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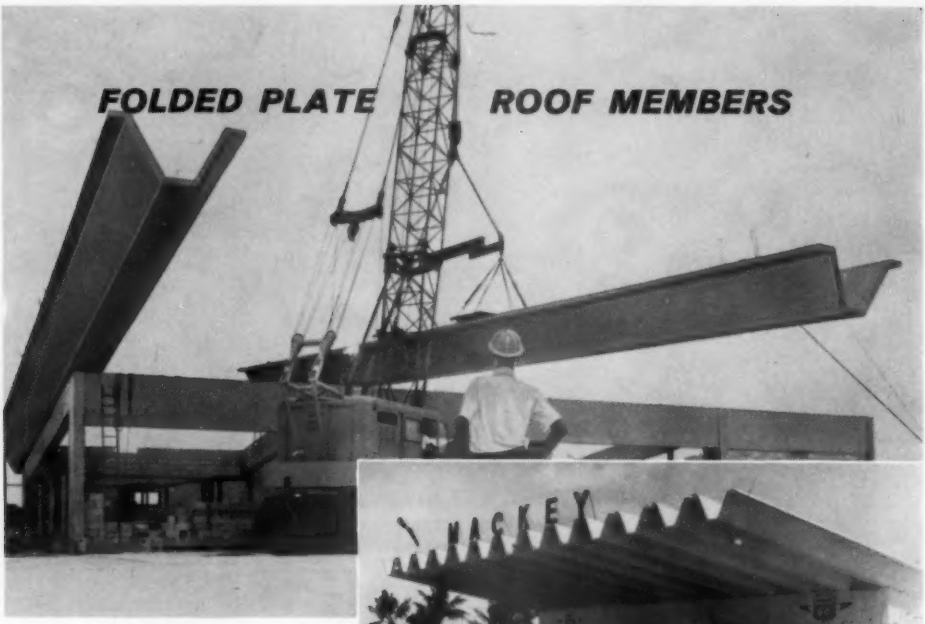

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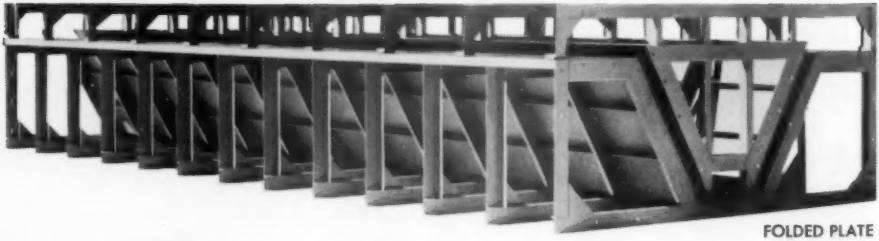

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December, 1961
Vol. 6, No. 4

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

The **JOURNAL** is published quarterly by the **PRESTRESSED CONCRETE INSTITUTE.**

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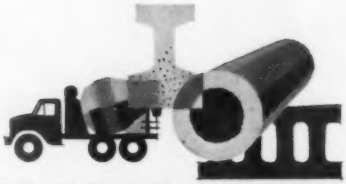

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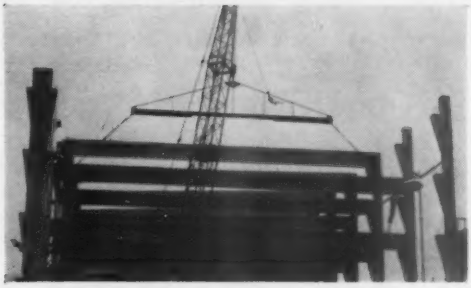

Selecting prestressed concrete construction for a new 5-story parking garage saved the City of Beverly Hills, California $186,000. This economy was made possible by using 75-foot prestressed concrete spans instead of conventional 30-foot spans of reinforced concrete. Use of Lin Tee spans eliminated interior columns, providing a continuous helical parking ramp and a 17% saving of floor area for the 400-car capacity garage. Erection of 24 pre-cast concrete columns and 77 Lin Tees was completed in only 18 working days. Columns and tees were pre-tensioned with **Leschen Prestress Strand.**

Prestressed concrete producers rely on Leschen for consistent quality prestress strand and technical information on tensioning materials. Let a Leschen Engineer discuss **your** prestress needs. Call, wire or write Leschen Wire Rope Division, 2727 Hamilton Avenue, St. Louis 12, Missouri.

Architect: Welton Becker & Associates, Los Angeles, Calif; **Consulting Engineers:** T. Y. Lin & Associates, Van Nuys, Calif; **General Contractors:** C. L. Peck, Los Angeles, Calif; Ellis E. White Co., Los Angeles, Calif.; **Columns & Tees:** Wailes Precast Concrete Corp., Sun Valley, Calif; **Prestressing Strand:** Leschen Wire Rope Division, St. Louis, Mo.

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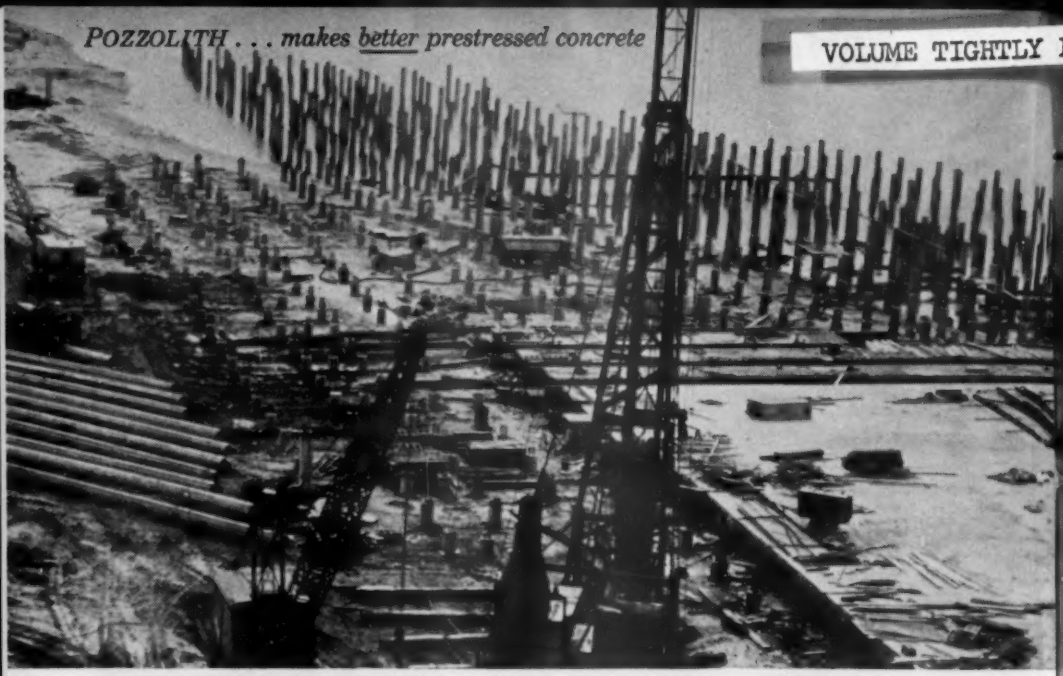

COLUMBUS STREET SHIP TERMINAL — Charleston, S. C. POZZOLITH concrete deck beams and 10″ concrete slab (lower right) are used on top of the pilings for 750,000 sq. ft. dock. Construction Agency: South Carolina State Ports Authority, Charleston • Engineers: Joint Venture of Lockwood-Greene Engineers, Spartansburg, S. C., and Palmer and Baker Engineers, Inc., Mobile, Ala. • General Contractor: J. A. Jones Construction Co., Charlotte • Prestressed Concrete and POZZOLITH Ready-Mix Concrete Supplier: Concrete Materials, Inc., Charlotte.

95 miles of prestressed piles

for new $7.7 million Charleston shipping terminal

Over 500,000 linear feet of 21-inch octagonal prestressed POZZOLITH concrete piling went into the foundation of the main dock of this new Columbus Street Ship Terminal. The 5,000 piles averaged 100 feet in length.

To keep the fast-moving pile-driving operation supplied with up to 30 piles per day—a compact, completely mechanized prestress plant was set up near the job site. Concrete was automatically batched, turbine-mixed and placed into 500 ft. casting beds.

The eighteen 7/16″ strands in each pile were tensioned to 18,900 lb. per strand. Piles were steam cured in the beds for 12 hours at 160F to a minimum of 4000 psi, then transferred to a 10-acre yard for 14 days of final curing. Using Type I cement, the zero slump mix was designed for 5000 psi at 28 days, but actual strengths averaged over 7000 psi.

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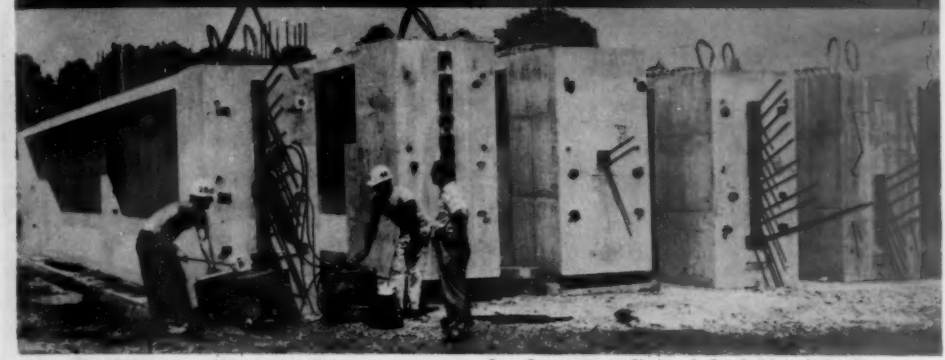

Post Office Annex, Philadelphia, Pa.
Architect-Engineer: Wallace & Warner, Bryn Mawr, Pa.
Structural Engineers: Urban Engineers, Inc., Phila., Pa.
Gen. Contractor: McCloskey & Co., Builders, Phila., Pa.
Concrete Precaster: Eastern Prestressed Concrete Corp., Line Lexington, Pa.

THE PROBLEM: How can a prestressed concrete products manufacturer economically cast girders of unusual design, not readily adaptable to existing pretensioning beds?

THE SOLUTION: Post-tension the girders with STRESSTEEL BARS.

This unusual precast structure, requiring up to 600 lbs. per sq. ft. live load, called for many short, deep girders with a wide range of prestressing forces as the photograph shows. To cast these girders in pretensioning beds was unfeasible for both cost and production reasons: the girders were too large for the beds—varying prestressing requirements would mean an uneconomical use of steel—unusual location and position of the steel center of gravity.

By casting the girders in another area and post-tensioning with STRESSTEEL BARS, Eastern Prestressed Concrete Corporation not only economically fulfilled the contract, but kept their pretensioning beds free for their regular production.

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Rapid construction, fire resistance, and spaciousness were the key factors in the choice of precast, prestressed concrete design for the new 250,000-sq ft Lynchburg Works of Delta Star Electric Division, H. K. Porter Company, Inc. Use of prefabricated concrete girders, beams, roof and floor units, and canopies helped meet a construction schedule of only eight months. On some days as much as 20,000 sq ft of double-T roof deck was erected.

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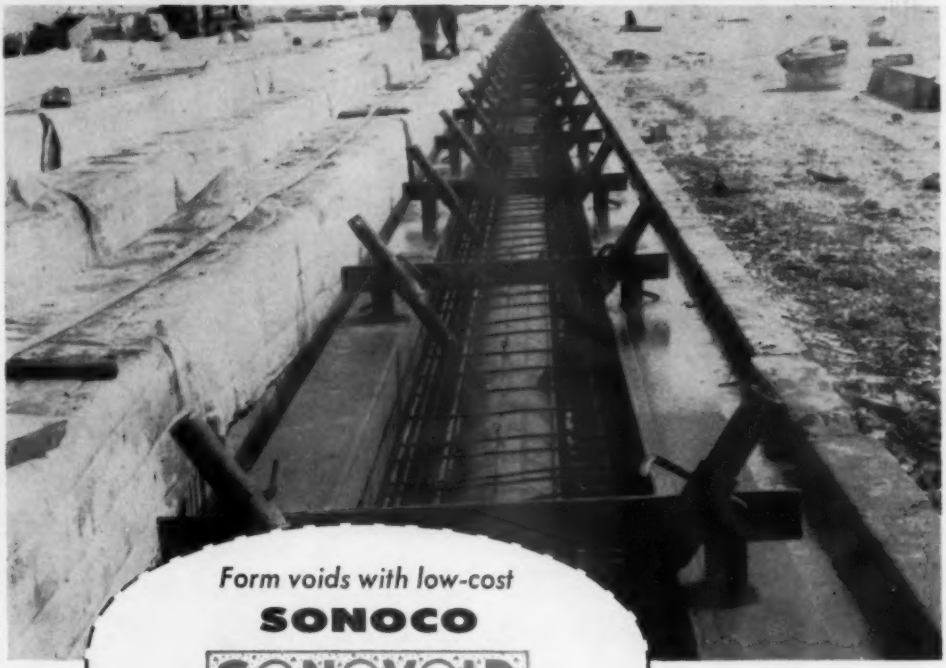

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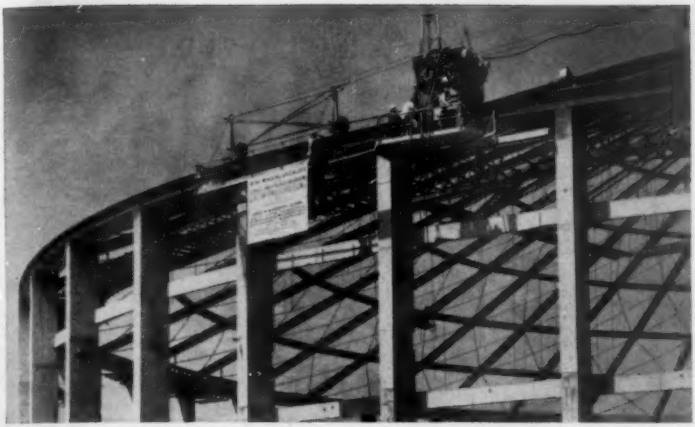
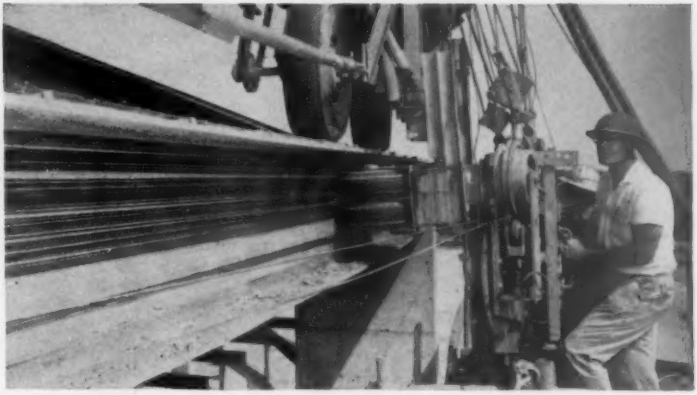


Your assurance of American-made quality

This is Mississippi's partially completed coliseum in Jackson. Visible atop the columns is the tension ring beam that supports the roof. Prestressed with CF&I Wire, the beam will withstand more than 3 million pounds tension.

160 miles of wire prestresses new coliseum



Wire-winding machine, operated by its designer, Mr. Jack Crom, pulls CF&I Prestressed Concrete Wire tight around the ring beam. After the wire has been laid and tensioned, pneumatic mortar is applied. In all, 15 layers of prestressing wire were used.

The State of Mississippi's new coliseum—now under construction on the Fair Grounds in Jackson—will feature an unusual form of construction that is both economical and efficient. The 2800-ton conical roof is supported by a giant tension ring beam prestressed with 160 miles of CF&I Prestressed Concrete Wire.

This prestressed method was the first choice because preliminary engineering studies proved a saving of approximately 10% could be realized in comparison with other methods.

Why was CF&I Prestressed Concrete Wire chosen? "Because we had dealt with CF&I in the past," Mr. Crom, President, Crom Prestressing, Inc., an affiliate of the Crom Corporation, Gainesville, Fla. says, "and they have always provided us with valuable technical information and helped us with our metallurgical problems. Also, we've always been able to count on prompt delivery of shipments."

In addition to Prestressed Concrete Strand and Wire, CF&I produces a wide range of steel products for American industry, including: Welded Wire Fabric; Rebars; Nails and Staples; Realock Chain Link Fence; Grader Blades and Cutting Edges; CF&I-Wickwire Rope; and many other quality steel products. For complete information, contact your nearby CF&I sales office.

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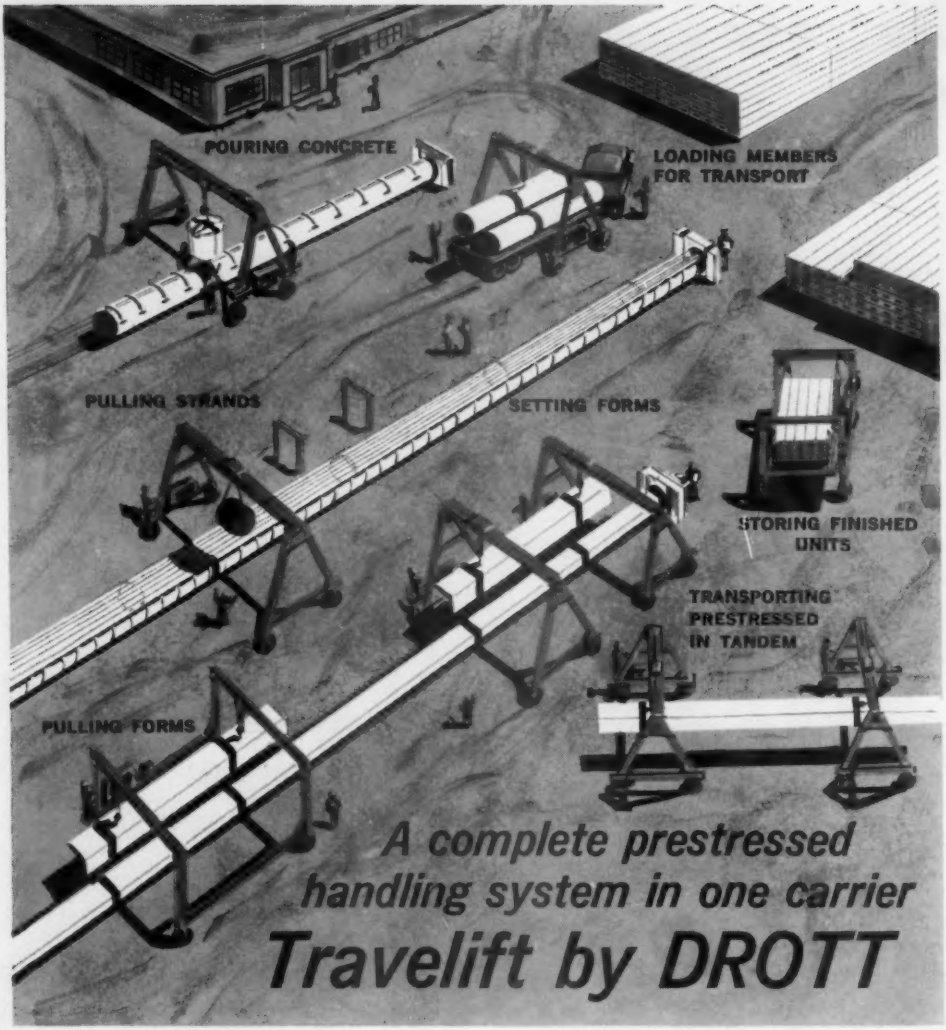

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The contractor cast the more than 350 girders adjacent to columns to simplify erection and eliminate overloading of the deck with heavy moving equipment. Tendon delivery was coordinated with casting, which averaged 3 girders per day.

Whether you are designing massive pier girders or a delicate shell roof, the Prescon System of prestressing increases freedom of design and choice of cast-in-place, lift slab, or precast construction. If you are not acquainted with the wide range of examples of prestressed concrete using the Prescon System, the Prescon representative will be glad to show them to you.

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Robert J. Lyman
President Prestressed Concrete Institute

PRESIDENT'S MESSAGE

It is with humbleness and optimism that I have accepted the charge of responsibility to serve as President of the Prestressed Concrete Institute. This office represents a challenge which, God willing, I intend to meet with your cooperation and assistance; through an active and forward administration guided by your needs and requirements.

Our industry must come to realities; the gleam of newness has rubbed off prestressed concrete, but that should not deter us since we still have a tremendous potential and a promising future. We are past the "New Frontier" stage and we are on the threshold of a new era which I predict will be the "Concrete 60's". During these "Concrete 60's" we can and will meet demands for striking shapes and at the same time promote the standardization which is so important.

The past year has seen many achievements take place; 5 successful fire tests, the completion of Project E.H.V., the launching of our fine advertising program, membership in the Reinforced Concrete Research Council, and a new spirit of co-operation between the PCI and organizations such as the AASHO, ACI and the AREA.

I have high hopes that 1962 will see the successful conclusion of our fire testing program, a continuance of our advertising program, and a further expansion of our publications effort pointed at placing in the hands of our Professional members as well as our Active and Associate members the most up-to-date engineering and production data available. It appears also that several important T.A.C. Committees will be able to finish their work early in 1962 which will make their standards and data available to all of us.

In looking to the future we must prove prestressed concrete's versatility in meeting the architects and engineers demands for products and shapes to permit maximum efficiency. We must also work to make the words "prestressed concrete" stand for and be synonymous with quality. I am also hopeful that in this coming year formal liaison with governmental construction agencies in Washington can be started.

Finally, let me say that my door will always remain open to all members whether it be by personal visit, telephone contact, or correspondence, for the discussion of affairs of the Institute so that its operation may truly be one of maximum benefit to all its members.

JOURNAL in brief

Design and Construction of Evergreen Plaza Shopping Center Using One Million Square Feet of Precast, Prestressed Floor Slabs 14

E. A. Picardi

The application of a new prestressed section to a job of this magnitude required careful engineering and production control. This machine made hollow slab is probably the only precast section prestressed in two directions. This coupled with its thin flanges makes the fabrication of this section a real achievement in concrete production.

Review of Two Prestressed Concrete Projects Built in Southern California in 1961 22

S. Galezewski

The meeting of the rigid specifications and tolerances required for a precast prestressed monorail beamway are thoroughly covered by the author. The ticklish job of adding four new levels of parking above an existing two story garage are explained and illustrated also.

A New Concept for Prestressed Concrete 36

T. Y. Lin

Three approaches to the design of prestressed members are presented. The load balancing concept is explained as a method of simplifying the design of prestressed members and structures. Several structures are illustrated as examples.

A Monolithic Post-tensioned Parking Structure 53

Carroll S. Delaney and Neil R. Runyan

A parking garage with features such as a 147° curved prestressed ramp beam and a post-tensioned dia-grid floor with precast concrete pans are described in detail by the author.

Research in Prestressed Concrete at the University of Florida and its Practical Application in Bridge Practice 60

W. E. Dean

This respected Florida bridge engineer outlines a few of the notable contributions to the knowledge of prestressing made possible by the excellent relationship between the University of Florida and the Florida Road Department. A listing of prestressed concrete research conducted at the University of Florida is also included.

Red Deer River Bridge 71

George Adam and D. de Wolf

Many engineers consider the precasting of bridge members to end at 80 ft. but the authors explain the fabrication, handling, and erection of precast prestressed girders, 148 ft. long and weighing 112 tons.

Design and Construction of Evergreen Plaza Shopping Center Using One Million Square Feet of Precast, Prestressed Floor Slabs

Presented at the Prestressed Concrete Institute Convention
October, 1961

by **E. A. Picardi***

The 8.3 million dollar Evergreen Plaza Shopping Center, at Evergreen Park, Illinois, is probably one of the largest totally precast and prestressed concrete shopping centers built to date. Approximately 900 precast "T" bents and 1700 precast, prestressed core slabs are being manufactured and erected to form the 500,000 square feet of supported floor and 500,000 square feet of elevated parking deck.

The project is an addition to an existing shopping center. The addition will consist of a new three-story department store, multiple smaller two-story shops, ground level and elevated malls, and ground level and elevated parking areas.

Figure 1 shows the new construction, floor areas, parking capacity and overall dimensions.

After considerable study, a basic bay size of 22 ft-0 in. x 49 ft-6 in. was found to be an adequate and economical solution for both the stores and parking structure. The structural elements forming the basic bay unit were developed to consist of poured-in-place footings and piers supporting precast concrete bents at 22 ft-0 in. centers with precast, prestressed concrete slabs spanning 49 ft-6 in. between bents. Special attention was given to minimizing the number of special precast units which resulted in use of only 4 basic bent forms, and a single basic spandrel beam form.

Typical construction details are shown in Figure 2.

The designers original intent was to span the 49 ft-6 in. between bents with precast, prestressed double "T" or "F" slabs. When bids were solicited for these slabs, the Material Service Corporation of Chicago proposed the use of an alternate floor system consisting of an 8 ft-0 in. wide by 20 in. deep cored slab spaced 11 ft-0 in. center to center. A 2 in. structural topping was poured atop the slabs which thickened to 3½ in. across the 3 ft-0 in. gap between the cored slabs.

At the time of this proposal the Material Service Corporation had developed a 12 in. deep cored slab which was in production and had been used on a number of small jobs. They proposed to build a plant and the equipment necessary to manufacture the 20 in. deep sections in the necessary quantity and within the specified time limits. They further proposed to engineer the product to meet the design specifications, inspection and load test requirements set up by the Architect-Engineer. The Material Service Corporation was permitted to compete with the double "T" and

*Chief Structural Engineer
Skidmore, Owings, & Merrill

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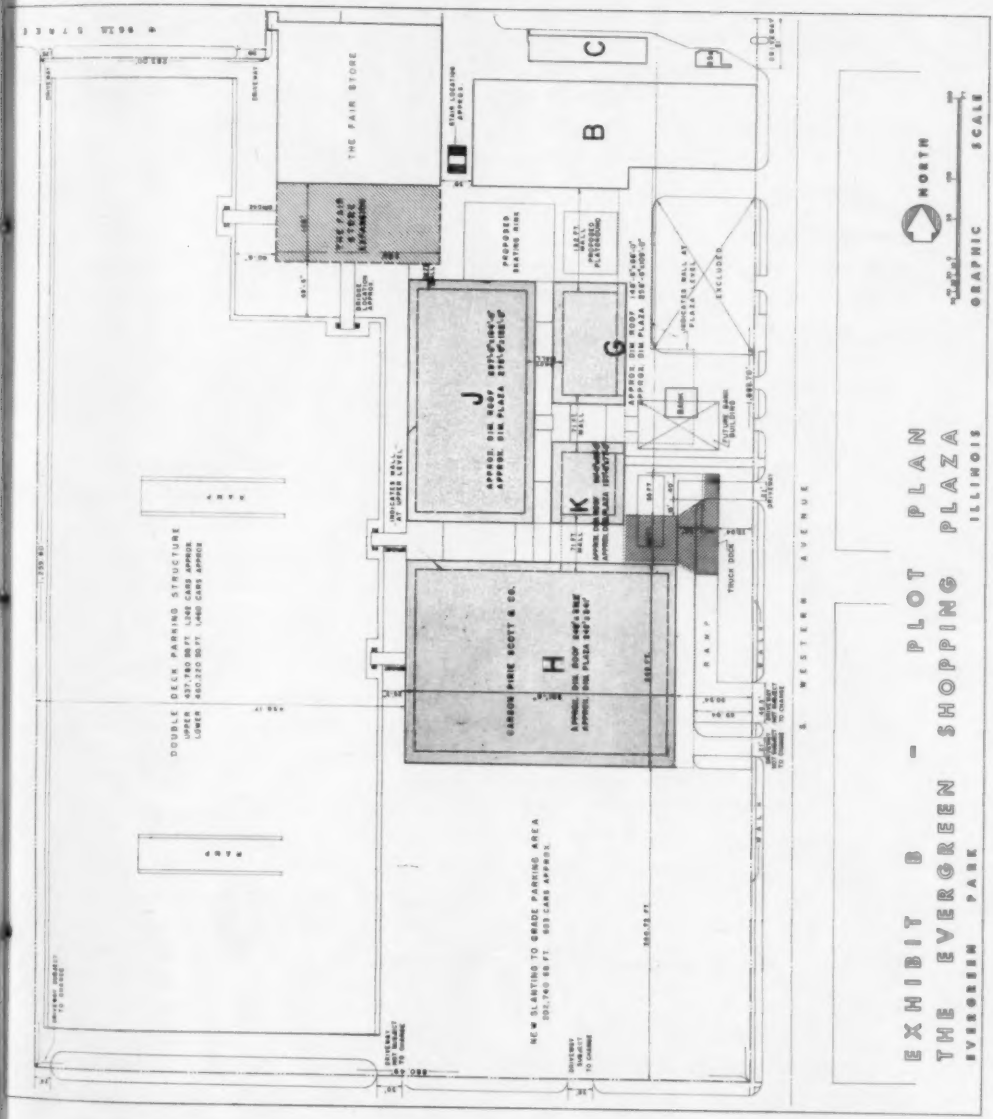

Fig. 1

VOLUME TIGHTLY BOUND

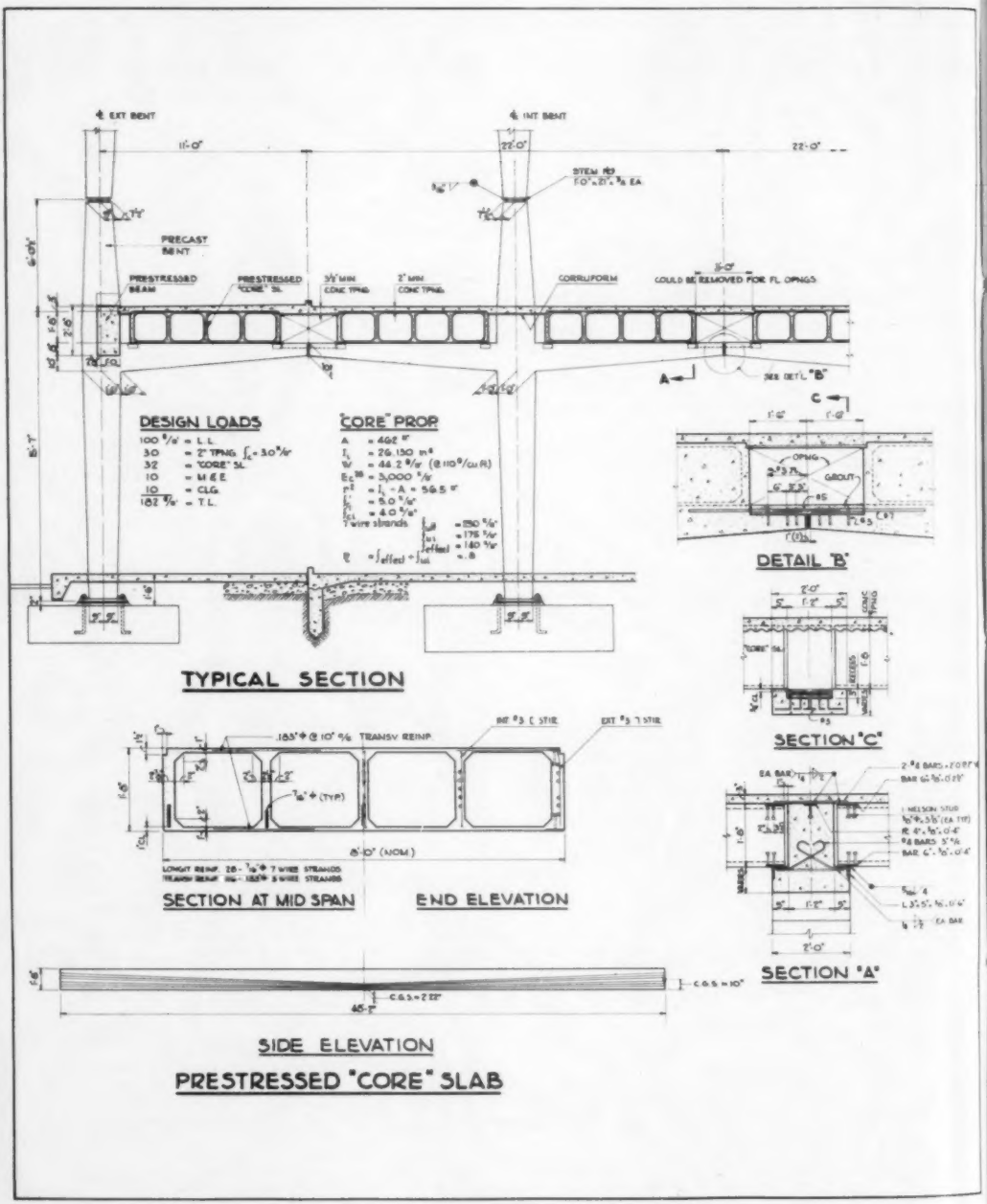

Fig. 2

"F" slab manufacturers on this basis. Approval by the Architect and acceptance by the developments owner of the Material Service Corporation's successful bid was encouraged by the ability of the firm to undertake a job of this size as evidenced by their physical size, financial strength, and engineering and manufacturing competence.

Thus, by late 1960 the project was well underway on paper, and construction of plant and equipment to make the prestressed core slabs was in progress. Figure 2 shows the cross section of the core slabs and side view of the prestressing strands.

A design live load of 100 pounds per square foot was used for all floors in the stores and for the outside mall areas. The parking deck was designed for a live load of 75 pounds per square foot. Provision in design was made for an additional deck on the parking structure and an additional floor over all shop areas except the department store. Thus, there are essentially three basic floor slab units, i.e.: garage slabs, mall slabs, and roof slabs for the department store. All units have the same concrete core slab section and dimensions. Variation in load carrying capacity is accomplished by variation in number and location of prestressing strands and in mild steel for shear reinforcing.

The 8 ft-0 in. x 20 in. core slab spanning 49 ft-6 in. has a flange thickness of 1 in. Exterior webs are 2⅞ in. thick and interior webs are 2½ in. thick. The section is constructed of 5000 psi light weight concrete using "Materialite" aggregate. The core has a cross sectional area of 462 square inches and a moment of inertia of 26,130 in.^{4}. The slab weighs 44.2 pounds per square foot. This design section produces a high moment of inertia for the amount of concrete area used in comparison to other long span slabs.

The 11 ft-0 in. composite floor section consisting of one core slab with a 2 in. stone concrete topping and 3½ in. thickened side toppings and weighs 62 pounds per square foot. It has a moment of inertia of 43,000 in.^{4}.

Using 28-7/16 in. diameter prestressing strands, the floor system is designed for a total DL + LL of 182 pounds per square foot and an ultimate load of 1.2DL + 2.4LL equal to 339 pounds per square foot. Camber of the core slab after release of prestress is approximately 1⅜ in. After the topping is poured, the camber of the structural system is about ⅞ in. and the total deflection under full LL is approximately 1 in.

Three official load tests to destruction have been made for the purpose of confirming design computations and proving workmanship and materials. The Architects specified that the slabs were to fail in flexure and not in shear. Failure was to take place at or beyond the ultimate load application.

Failure in load test "A" occurred below ultimate load. There was a progressive development of horizontal cracking mid-way across the web to top flange fillets, starting at the ends of the webs and extending toward the center of the span. This test also showed excessive development of diagonal shear cracks in the end 3 to 5 feet of the webs. The horizontal splitting in the top fillet flange to web connection indicated improper anchorage of the stirrups, and the excessive diagonal cracking in the ends of the webs indicated need for closer stirrup spacing. These corrections were made and official acceptance of the slab was given as a result of its performance in test "C".

The slab used for test "C" had

closely spaced web reinforcement in the ends of all ribs. The stirrups in the exterior ribs were bent 2 ft-0 in. horizontally into the 1 in. top flange. In the interior ribs they are hooked 2 in. into the top flange with hooks turned in the longitudinal direction.

Before the test began, existing hair-line cracks, all of which were minor shrinkage cracks, were marked on the slab. As load was applied in increments, new web cracks did not appear before design DL + LL was reached. Design DL + LL is approximately 41% of the ultimate load when the slab is expected to yield in flexure.

Flexure cracks near mid span began to appear at about 50% of ultimate load. At about 70% of ultimate load, the center web at the north end began to develop a number of diagonal cracks similar to those in previous tests indicating local overstress. These cracks began to open and travel. At 90% of ultimate design load they were wide enough to be considered a local failure. However, all other webs held remarkably well with no evident widening of diagonal cracks.

As the load approached the ultimate, the slab started to yield in flexure showing pronounced deflection with very little increase in load.

Final collapse occurred at a deflection of 23 in. with a total applied load of 142.5 kips representing 102% of design ultimate load. At collapse all the webs were in place and failure was due to simultaneous crushing of the top flange at two points in the middle third of the span.

To determine the cause of the difference in shear resistance in the interior webs at one end of the slab, the web reinforcing was exposed after collapse. The web reinforcing in the center rib (which showed the first and only serious cracking) was found to be slightly lower than in adjoining ribs.

This further emphasized the sensitivity of this slab to variation in di-

Fig. 3—Shear failure in test "A".

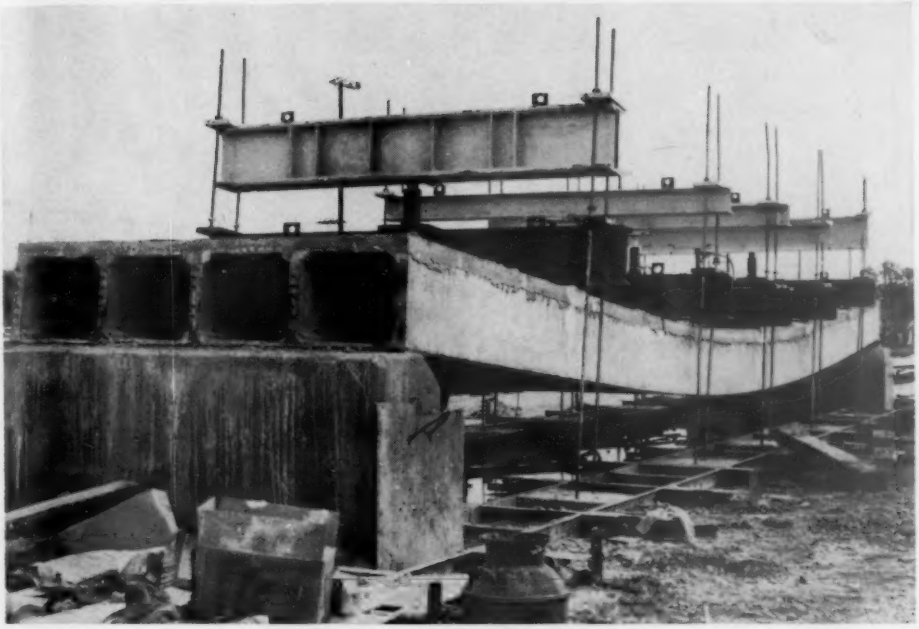

Fig. 4—Deflection at ultimate load for test "C".

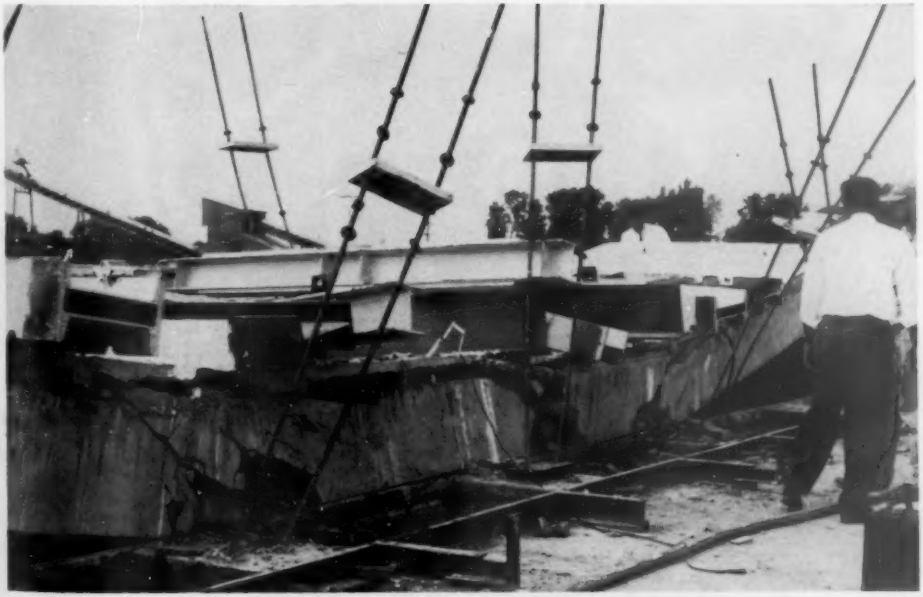

Fig. 5—Member after failure at ultimate load for test "C".

mensions, placement of reinforcing and proper anchorage of stirrups into the top flange.

Final design of these important stirrups will probably consist of inverted "U" shaped members placed as high in the slab as possible.

Figure 3 shows the shear failure occurring in load test "A". Figure 4 shows the slab loaded to ultimate just prior to the flexure failure in load test "C" and Figure 5 shows the slab after failure in load test "C".

We feel that this project should be of interest to the engineering profession and the prestressed concrete industry, because it is one of the first attempts in the United States to produce a prestressed concrete slab with very thin flanges and webs. We believe such sections are extremely sensitive and require precise equipment for their manufacture, careful attention to accurate placing of all steel, and optimum quality control and inspection.

New sections of this type must be carefully designed and fully tested prior to acceptance. We believe that this can be done and that this and similar types of slabs can be economically produced in fully mechanized plants.

We wish to acknowledge the work of: Messrs. A. Brown and L. Maskaliunas and Dr. F. Khan of the firm of Skidmore, Owings & Merrill, Architect-Engineer, for the project. Mr. C. Goodman and C. Rosenstern of Material Service Corporation, subcontractor for the concrete work. Mr D. Byrnes, of George A. Fuller Company, the General Contractor for the project and Mr. J. Janey, who supervised the physical testing program.

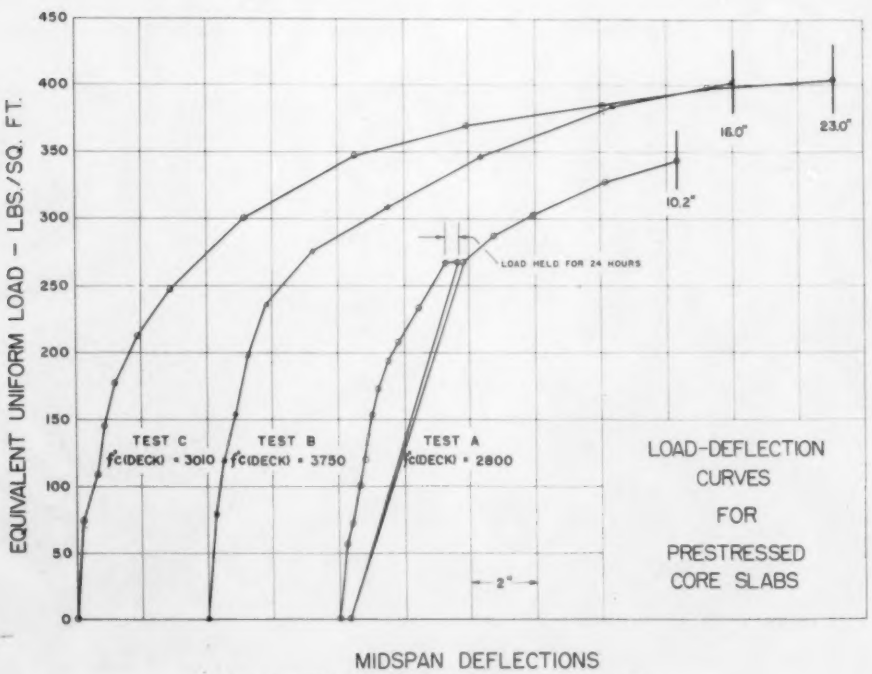

Fig. 6—Load-Deflection curves for tests A, B, & C.

Fig. 7—Photograph of model of completed plaza.

VOLUME TIGHTLY BOUND

PROCEEDINGS PAPER

Review of Two Prestressed Concrete Projects Built in Southern California in 1961

Presented at the Prestressed Concrete Institute Convention
October, 1961

by **S. Galezewski***

In recent years prestressed concrete has won its place as the material which is most successfully meeting the needs and demands of the growing construction programs. It has been proven many times over, that prestressed concrete when intelligently used, gives the engineer or architect an excellent solution to many problems. Two such solutions are reviewed in this paper. It is hoped that these brief discussions will stimulate the curiosity and imagination of men who, in the years to come, will undoubtedly contribute to the field of prestressed concrete their own dreams in the form of workable designs.

First Prestressed Concrete Monorail

Many large cities of today are faced with the problem of mass public transportation. City Planners, business leaders, architects and engineers are well aware that we must find some rapid transit system which will extend the commuting distance from the center of the city, so that a commuter can travel sixty or seventy miles in less than an hour. Unless we do this, traffic congestion and deterioration of city centers will continue very rapidly. They also know that trying to plan a program for the next five or ten years is almost a waste of time. Such facilities would be out of date before they were completed.

Numerous comparisons with standard supported two track railways and other means of communication indicate that prestressed concrete monorails can give a superior, less conspicuous, and noiseless structure. Light weight cars on pneumatic tires can provide swift, comfortable and silent transit.

In several major population centers monorails are much closer to construction than the general public realizes. Their safety and dependability have already been proven in the monorail under discussion.

Prototype of the Future System

The Disneyland Monorail System may be called a prototype of the supported monorail readily adaptable to mass public transportation. This system, now two and a half miles long, installed at a cost of $3,300,000 has been in successful operation for two years. Over five million passengers have been carried. The monorail parallels a major highway in Anaheim, and twice crosses high over a city street. The trains receive and discharge passengers at an elevated station at the Hotel, as well

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Rockwin Engineers and
Rockwin Prestressed Concrete Corp.

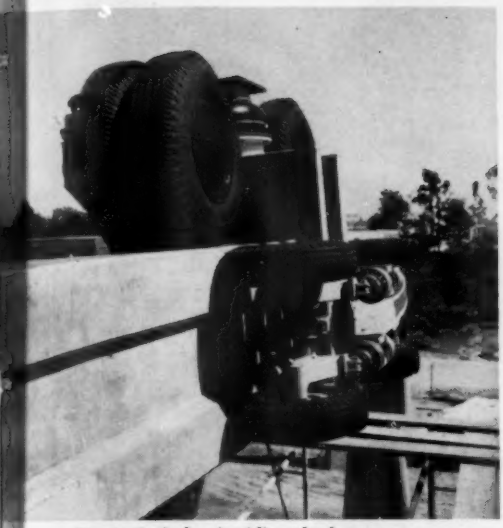


Fig. 1—Train bogie riding the beamway.

as at a station within the park. It is the longest, modern passenger-carrying monorail system in the world. Experiences over the past two years have indicated that this type of transportation system has public appeal and public acceptance. At the present time the three four-car monorail trains are carrying in excess of twenty thousand passengers per day.

Supported Type Monorail

In the supported type system, the train carriages or bogies roll on the top surface of the beamway and grip the sides of the beamway for lateral stability. (See Fig. 1). For comparison consider the suspended monorail system, where the train is hung below the beamway and is stabilized by wheels rolling along the sides or by some form of gyroscopic action. The supported system such as used at Disneyland resulted in lower structure height, which effected better esthetics and lower cost. Additionally, this system provides a positive lateral stability of the train—free from the pendulum sway of a suspended train.

Trains

The 26 ton trains consist of four cars with articulated bodies. Each train has sixty high pressure, pneumatic rubber tires grouped together in five bogies. Each bogie consists of two sets of dual 8.25 x 20 tires which roll on the sides of the beamway. At both the second and fourth bogies of each train are mounted two series-connected 300 volt, 55 H.P., D.C. traction motors, providing 220 H.P. drive for each train. The high friction value of rubber on concrete combined with the high torque at low R.P.M. of the D.C. traction motor, result in rapid acceleration from the stations. Electrical energy is supplied by two steel and copper bus bars mounted on one side of the beamway. Four 600 ampere silicon diode rectifiers provide 600 volts at the bus bars. The trains will accelerate at two miles per hour per second easily. The 6.72% maximum grade for the Disneyland Monorail is substantially under the maximum grade capabilities of the system. The electrical power system and the pneumatic tires on a concrete running surface result in trains that are almost noiseless in operation.

Beamway

Design and construction of the

Fig. 2—Preparation of beams prior to form placement.

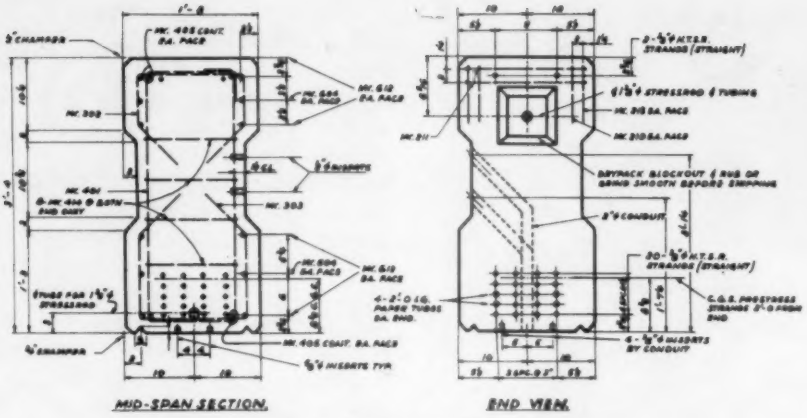


Fig. 3—Prestressed beam details.

recent one and three-quarter mile extension of beamway to the Disneyland system was accomplished in seven months. Careful engineering and close field control resulted in a precise alignment for all components of the beamway. Costs of the beamway including foundations, pylons, beams and complete electrical power system approximated $1,352,000 for 8700 feet of extension. This indicates a cost per mile of $820,000. for this particular installation.

The prestressed concrete portion of the project had an extremely tight schedule. Rockwin Engineers designed the beams and secured all the pertinent approvals within three weeks time. Rockwin Prestressed Concrete Corporation produced 3000 lineal feet of beamway in the following four weeks. This production record is remarkable in view of the fact that it was achieved under rigid specified tolerances.

Prestressed Concrete Beams

The prestressed concrete beams (See Figs. 2, 3, & 4) were produced on 420 ft. long prestressing beds in Rockwin Prestressed Concrete Corporation's prestressing yard in Santa Fe Springs, California. The sixty foot beams were pretensioned with 22-⅜ in. diam. H.T.S.R. strands and post-tensioned with 1-⅜ in. diam. rod placed inside a metal tube and draped to a parabolic shape. All strands were straight and fully bonded except for some strands which had a bond breaker for a short distance close to the ends. This was used as an economical means of raising the center of gravity of the prestressing force, in the end zones of the beams.

The main reason for the use of partial post-tensioning was to better control the uniformity of camber of individual beams. Concrete strength

Fig. 4—General view of prestressing beds and storage area.

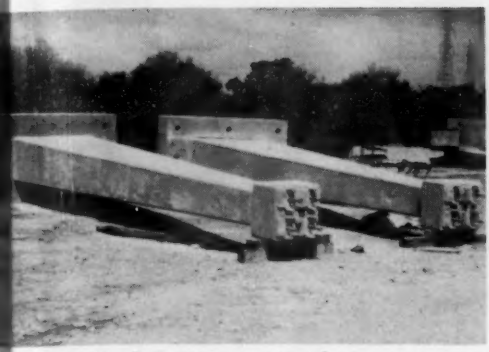


Fig. 5—Precast concrete pylons.

specified for the beams was a minimum of 5000 p.s.i. in 28 days. The actual strength obtained in 24 hours with about 10 hours of steam curing at 140°F. was 4300 p.s.i., giving a 28 day strength of 7000 p.s.i.

The service requirements of the beams imposed unusual camber, deflection, and tolerance requirements. These were specified and met by the manufacturer under the penalty of rejection if not strictly adhered to. The beams were designed, cast and tensioned in such a way that the final remaining camber of all the members at the time of erection was ⅜ in. plus or minus ⅛ in. In addition, the members had to be designed in such a way that the maximum deflection under applied loads, including impact, would not exceed ¾ in. at any point on the member. These conditions required unusual care on the part of production crews.

Camber can be affected by many factors. The most common source of camber variation is probably inconsistent concrete strength. For this reason, it was absolutely essential to keep a complete uniformity in concrete proportioning, mixing and curing. Inconsistent distribution of heat from one end of the bed to the other can be responsible for significant variations in concrete strength and, consequently, variation in camber. Recording thermometers were used at each beam to assure uniform curing. Another common cause of camber variation is non-uniform tension in the strands. Absolutely no variation from C.G.S. locations shown on drawings and from prestressing forces indicated was allowed on this job. Particular attention was also paid to the way the beams were stored. Careless storing without regard to bearing location can account for significant spread in camber. To keep the camber growth uniform and at a minimum, all the units were supported at the very ends with the distance between supports identical for all units of the same span.

The beams had to have an architectural finish, hence extra care was given to the finishing of the surfaces. The top surface was a steel trowel sweat finish. Any flutes along chamfered edges or around castings were removed by stoning, disc sanding or grinding. Imperfections such as rock pockets and broken edges were filled with epoxy-sand mortar and disc sanded to a smooth surface. All strands were burned flush in a formed recess, and the recess was filled with epoxy-sand mortar and disc sanded flush again.

Tolerances specified and achieved

Fig. 6—Forming the pile cap.

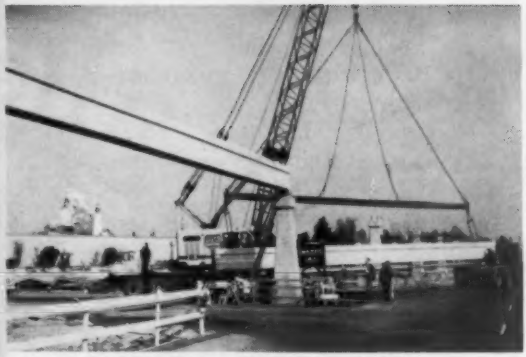


Fig. 7—Erection of a prestressed girder.

were of a magnitude unheard of in the normal production of precast or prestressed members.The sixty foot length of the girder had to be guaranteed to ⅛ in. plus or minus. This called for an investigation of axial shortening due to shrinkage, creep and elastic deformation. After long term effects were established, the proper allowance was made in the forming of the girders. Furthermore, the profile dimensions indicated on the drawings had to be guaranteed to be within 1/16 of an inch at any point on the girder. The permissible lateral deviation from an absolutely straight line was ⅜ of an inch in the entire length of the girder, or ⅛ of an inch in any single 10-ft. increment of length. These last two requirements made it essential that the forms be constructed from heavy sheet steel, free from undulations and without lap seams or joints. Joints in sheets were welded and ground, and forms were adequately braced with structural sections.

Girder Supports

The substructure supporting the girders consists of precast concrete pylons (See Fig. 5) which stand as high as 32 feet. Each individual pylon is supported by three 24 in. diam. poured-in-place friction piles that average 30 feet in length. The heads of the piles extend into the 9½ ft. square and 2½ ft. thick pile cap. (See Fig. 6). The anchor bolts were placed precisely so that minimum adjustment was necessary in the pylon alignment. The erectors very carefully placed and shimmed the girders into position with the aid of a hydraulic jack. The pylon to girder connection is adjustable and can be readjusted should settling or shifting occur.

Conclusion

This job has been a challenge to the Engineers and Contractors. The satisfaction of successfully accomplishing the task on time has been a great reward to all who took part in this unusual project. Although the Disneyland Monorail is not a city wide rapid-transit system, the only way to answer all questions pertaining to this subject is to build one. The experience gained in designing, building and operating the Disneyland Monorail should be of great value to those who wish to evaluate this type of system. Valuable data has been obtained which may be applied to any public rapid-transit system.

Fig. 8—Closeup of the joint detail showing the required precision tolerances.

Fig. 9—View of the straight portion of the beamway.

Fig. 10—Trains in operation on the prestressed beams traveling at 45 m.p.h.

First: Construction problems and construction costs were determined.
Second: The advantages offered by prestressed concrete members were definitely established.
Third: Trains were developed and physically tested in almost continuous operation.
Fourth: Information relative to maintenance was accumulated.
Fifth: Public reaction to this type of transportation was evaluated.

BEVERLY HILTON GARAGE, BEVERLY HILLS, CALIF. THE LARGEST HOTEL PARKING FACILITY IN THE WORLD

Expansion of any existing facility, unless it is planned ahead and provided for in the original design, is usually a problem. Prestressed concrete provided a good solution on this project. A 500 automobile parking structure, consisting of four new levels, was added atop the existing two-level parking structure. The garage now provides parking space for a total of 1500 cars.

Description of Structure and Erection

The garage is located in a part of town habitually congested by traffic. The jobsite was well hemmed in,

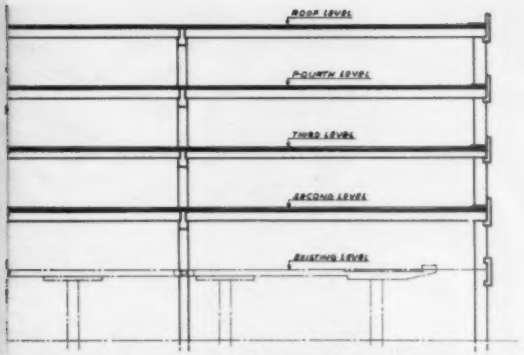


Fig. 11—Elevation of the Beverly Hills Parking Garage showing the existing structure and the new addition.

virtually inaccessible on three sides to construction equipment, and the working space was very limited both inside and around the structure. Under these circumstances, wide usage of precast units and erection from within was almost mandatory.

It was decided to shore the existing deck with heavy timber and permit the erection cranes and loaded pull-dollies to operate on the deck. Crane outriggers were supported on long 12 x 12 timbers to spread the weight over as large an area as possible. The upper surface of the deck was clearly marked to indicate the location of the shores below so that a maximum number of shores were utilized during any lifting operation.

The new addition rests on a completely independent foundation which was designed as a "continuous beam on an elastic foundation". The new footings are located next to and somewhat below the old footings. The main supports of the three bay, 160 x 266 ft. garage addition are 54 ft. long precast columns. The columns were gently lowered by crane through the cutouts in the existing deck to the new foundations. Every column was positioned on the upturned ends of four embedded bolts, with leveling nuts to allow for precise adjustment. To hold each of the columns perpendicular and in exact alignment with others until construction had progressed sufficiently, oak wedges were driven around all four sides of the 4-ft. square cut-out. Tilt-up type bracing was also installed from the columns to the existing deck.

The fascia's of all four additional parking levels and part of the existing second level were composed of continuous rows of precast prestressed spandrel beams. There were close to one hundred spandrels ranging in spans from 30 to 60 feet. They were 4 ft. 6 in. deep, and weighed up to 16.5 tons each. The spandrels function as structural members supporting the outside bays and also are purposely deepened to conform to the architect's appearance require-

Fig. 12—Completed erection of a line of precast columns. Note the hole cut in the existing floor and the wedges holding the column in position.

Fig. 13—First deck partially erected.

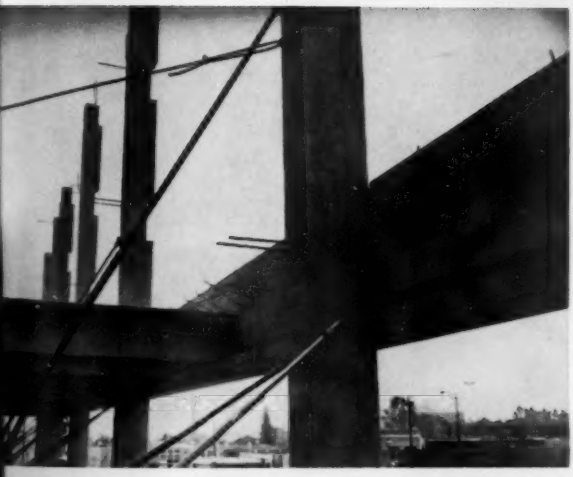


Fig. 14—View of the exterior precast columns and the spandrel beams.

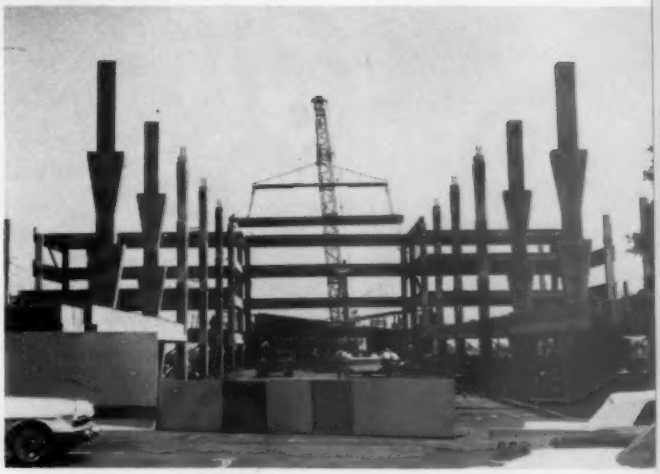


Fig. 15—Structure partially erected.

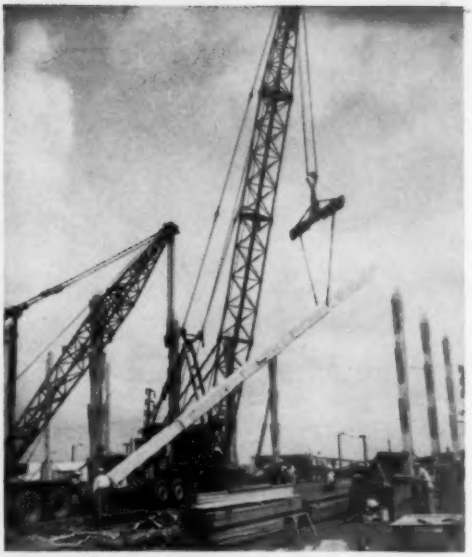


Fig. 16—Erecting a precast column.

Fig. 17—Guiding a column through the hole provided in the existing floor.

Fig. 18—Installation of the Oak wedges and braces.

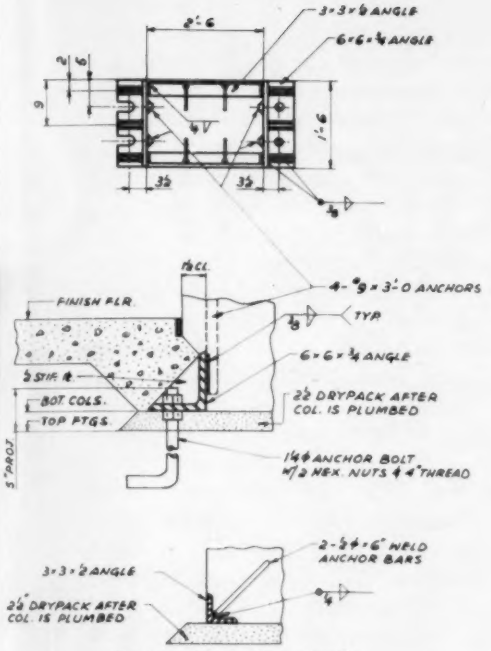


Fig. 19—Column base details.

ments. At the base and on the inside face of the spandrels is a concrete shelf which parallels a corresponding ledge in an inverted tee beam. The inverted tee beams span between all intermediate columns. They are pretensioned with 32-7/16 in. diam. H.T.S.R. strands, are 28 ft. 6 in. long and 34 in. deep. 72 beams of this type were erected in this structure.

The bulk of the prestressed members used in the project were more than 300-8 ft. wide double tees ranging in span length from 37 to 60 ft. The average weight of the double tee elements was about 13 tons. The floor system was comprised of the 8 ft. wide double tee slabs which were 22 in. deep spaced 10 ft. on centers.

Severe space restrictions complicated the usually simple erection procedures. Placing of double tees in the first bay, for example, was considerably more involved than would be the case under less restricted conditions. Tractor and pull-dolly combinations delivered two members per load, each truck backing out onto the old deck to where a 35-ton crane was positioned. After a sling had been attached to four pickup points, the beam was raised, rotated 180° to get around a column, and swung out into the respective bay. At this stage a second 35-ton crane outside the building was brought into use, hooking onto the beam's outer pickup points, with the other end of the tee temporarily resting on a cross member. The sling was slacked off, to be reattached to the inside end only. Finally, both cranes operated together to place the tee in exact position between supports, where an erection clearance of 1 in. was allowed. Several hundred members were erected by Rockwin Prestressed Concrete Corporation in record time without the loss of a single member.

Design Considerations

Although most of the precast prestressed elements used in this structure are fairly standard throughout

Fig. 20—Close-up of a completed column base connection.

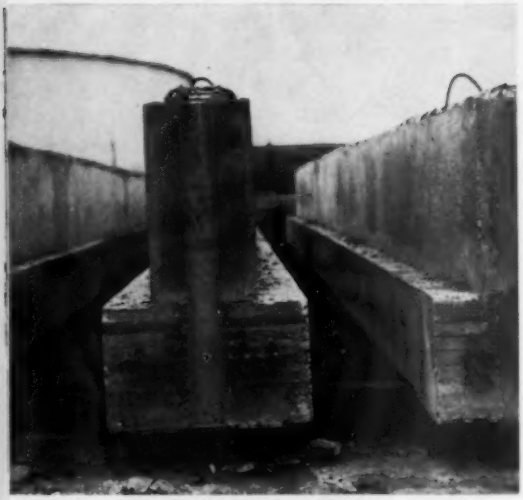


Fig. 21—Inverted tee beams.

the industry, it was a West Coast debut for an 8-ft. wide double tee. The design approach was also somewhat unusual. The 8-ft. double tees were placed 10-ft. on centers and the 2-ft. wide strip between the tees was poured monolithically with a 2½ in. topping. The topping served as a diaphragm and its function was to transfer all seismic loads to properly located shear walls. After the topping has reached its full strength, the composite 10-ft. wide section will carry all superimposed loads. All prestressed pretensioned double tees were stressed with 24-7/16 in. diam. H.T.S.R. strands and were harped at midpoint. Furthermore, all tees on this project were designed for live load continuity. This was attained by anchoring mild steel bars in the ends of the tees and projecting the bars into the poured-in-place concrete where additional mild steel bars were placed in the zones of negative moment. (See Fig. 23). The inverted tee beams were also designed for live load continuity and special sleeves were cast in the precast columns to permit the negative beam steel to pass through the columns. (See Fig. 24). A somewhat different approach was used to obtain the same live load continuity in the spandrels. (See Fig. 28). Here, after specified concrete strength of all poured-in-place concrete was reached, a special transition plate was placed on top of the plates of the two adjacent spandrels and was field welded, developing the required continuity. After field welding, the pockets were drypacked with Embeco to protect the steel plates from corrosion, giving the members a uniform and continuous look. It was recognized that due to a certain amount of creep the assumed live load continuity would be somewhat nullified. Proper allowance in the stresses was made to counteract this long term creep effect on continuity.

The pre-established location of the elevator shafts and stairwells which were utilized as shear walls was not symmetrical and required an elaborate seismic analysis. (See Fig. 29). A number of struts and "drag beams" (See Fig. 30) which transferred the lateral forces to the stems of the double tees were incorporated with each structural diaphragm. Although the question of differential settlement between the two structures, if tied together, was recognized, Rockwin Engineers did not believe this to be sufficiently important to warrant giving up the ad-

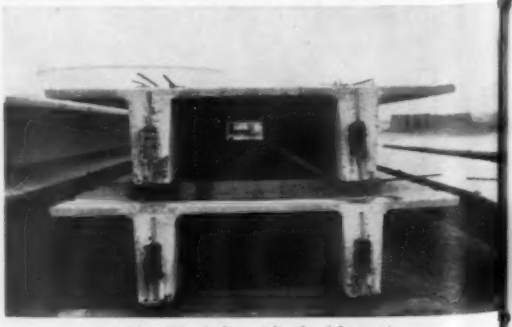


Fig. 22—8 ft. wide double tee's.

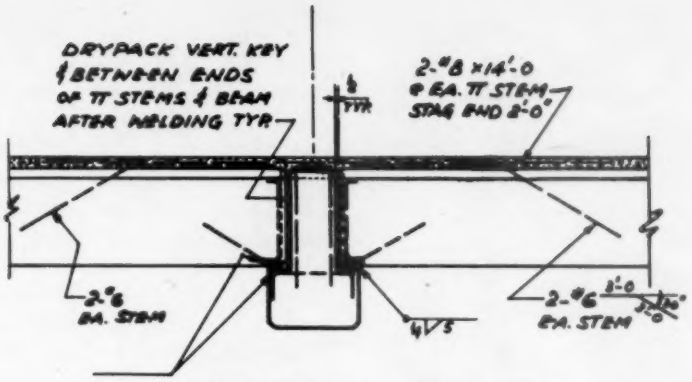


Fig. 23—Detail of double tee to beam connection.

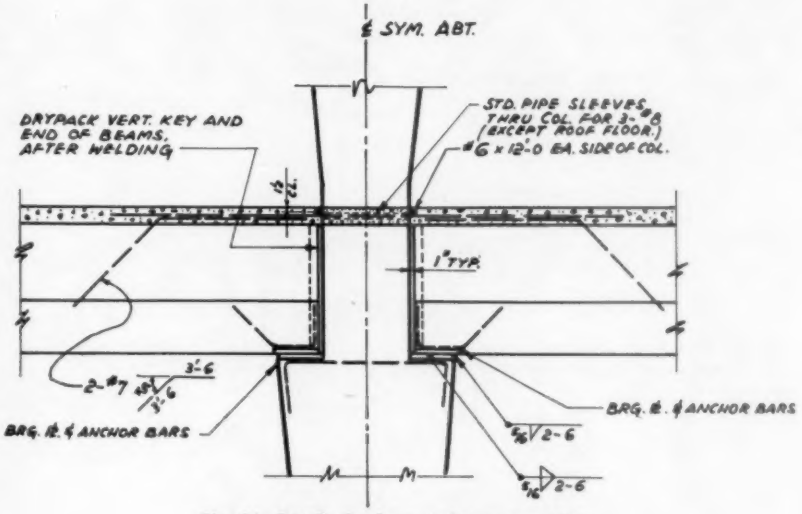
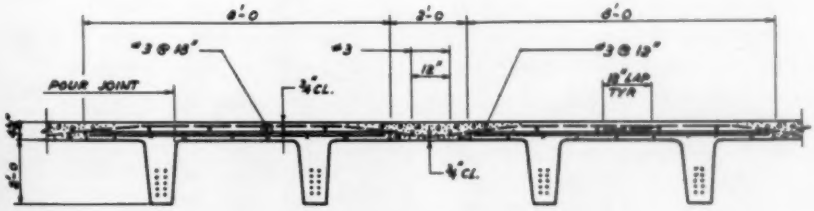
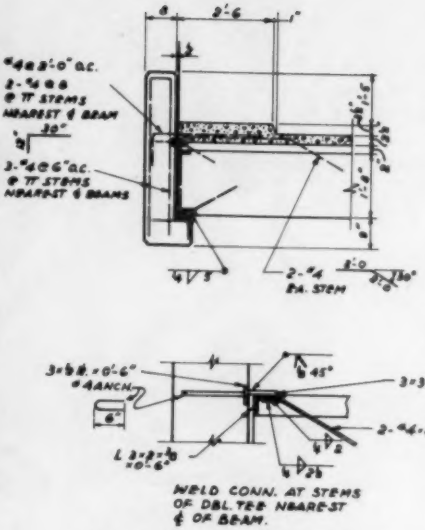


Fig. 24—Detail of column to beam connection.



NOTE: SLAB IS TO BE POURED ACROSS BLDG. IN WIDTHS NOT TO EXCEED 20'-0" ALLOW A MIN. OF 24 HRS. TO ELAPSE PRIOR TO POURING ADJ. STRIP.

Fig. 25—Cross-section thru floor showing reinforcing of topping above and between double tee units.



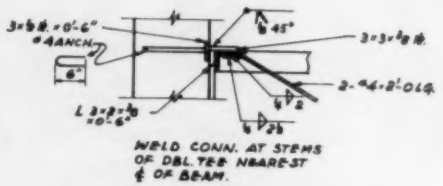


Fig. 26—Detail of continuous connection between end of double tee and spandrel.

vantage of increased rigidity, particularly in view of the comparatively light live loads involved and also in view of the fact that most of the settlement will take place during the construction of the building. After all poured-in-place concrete had been placed and had reached its required strength, additional steel was placed in the cutouts between the old slab and the new precast columns. The cutout was then filled with non-shrink concrete tying the old and new structures together.

It should also be mentioned that part of the roof of this garage was designed as a heliport. The long span 8-ft. double tees carried this unusual loading without difficulty.

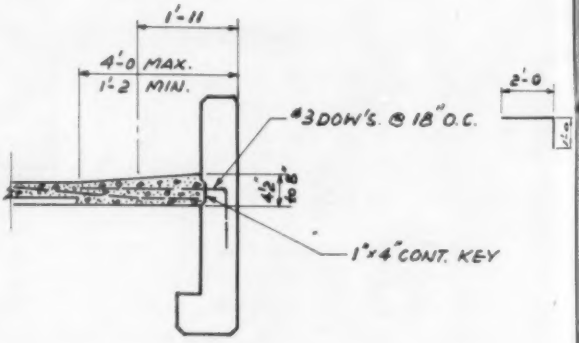


Fig. 27—Detail of connection between edge of double tee and spandrel.

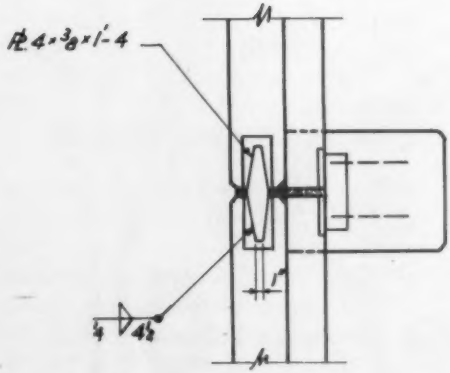


Fig. 28—Spandrel beam continuity detail.

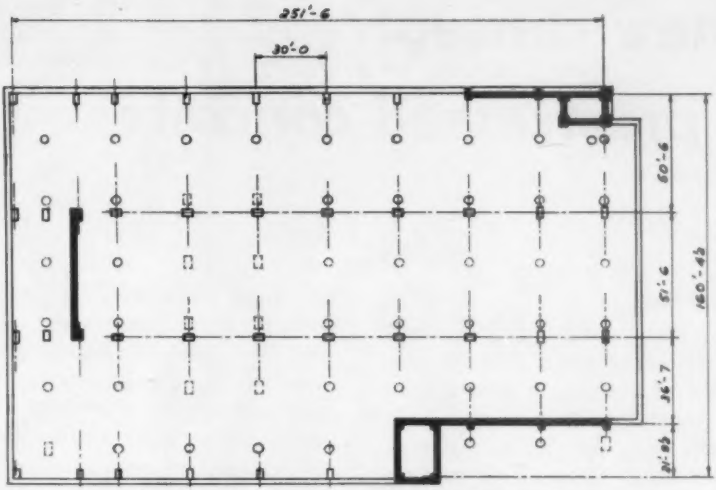


Fig. 29—Plan showing location of shear walls.

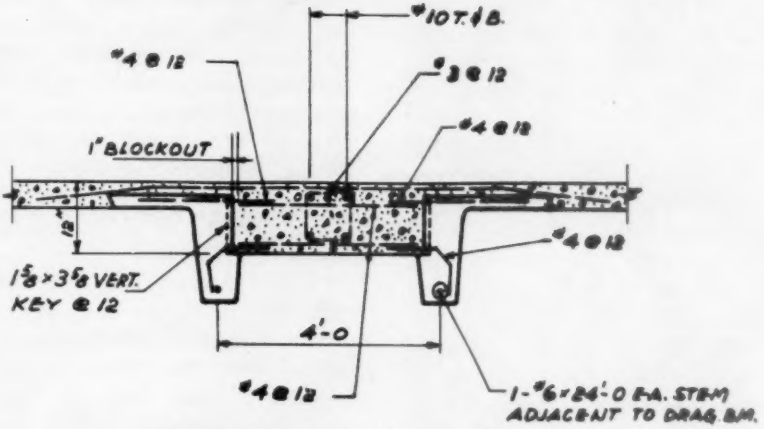


Fig. 30—Detail of drag beam used in seismic calculations.

Conclusion

Utilization of continuity through the mild steel reinforcing, as well as designing for composite action, resulted in a very economical framing system. The search for the most economical section for this garage created a new section—the eight foot wide double tee.

The contract price of this 170,000 square foot addition was $800,000., including elevators, screens and architectural treatment. Welton Becket & Associates were the architects for the project. The cost of the structural frame erected in place including columns, beams, spandrels, double tees and all other precast structural elements was $303,000.

Our experience on this project has not been without frustration, but the satisfaction of exploring relatively new avenues in design and construction, which resulted in a structurally worthy and economical building has been a great reward.

A new concept for prestressed concrete . . .

By T. Y. Lin*

Introduction

When prestressed concrete was first conceived, essentially by Eugene Freyssinet of France, it was visualized as the transformation of a brittle material into an elastic material. Concrete which is weak in tension and strong in compression was precompressed by steel under high tension so that the brittle concrete would be able to withstand tensile stresses. Thus prestressed concrete was dealt with in terms of internal stresses, with "no-tension" being the general criterion for design and construction. This approach might properly be termed the "stress-concept.'

As prestressed concrete became widely produced and adopted, a second concept was formulated, commonly known as the ultimate strength theory. Under that concept, prestressed concrete is treated as a combination of high strength concrete and high strength steel, with concrete to carry the compression and steel to carry the tension. Design formulas and code requirements were proposed as a result of that "strength-concept", and they were basically similar to those of conventionally reinforced concrete.

More recently, a third concept has been developed, chiefly by the author, but undoubtedly also utilized by other engineers. In the overall design of a prestressed concrete structure, prestressing is primarily intended to balance a portion of the gravity loads so that flexural members, such as slabs, beams and girders, will not be subjected to flexural stresses under a given loading condition. This transforms a member under bending into a member under axial stress and thus greatly simpifies the design and analysis of otherwise complicated structures. For convenience in discussion, we will term this the "balanced-concept."

Life-History of Prestressed Member Under Flexure

The life-history of a pre-stressed member under flexure is briefly shown in Fig. 1. There are several critical points as follows:

1. *The point of no-deflection which usually indicates a rectangular stress block across all sections of a beam.*
2. *The point of no-tension which indicates a triangular stress block with zero stress at either the top or the bottom fiber.*
3. *The point of cracking which generally occurs when the extreme fiber is stressed to the modulus of rupture.*
4. *The point of yielding at which the steel is stressed beyond its yield point so that complete recovery will not be obtained.*
5. *The ultimate load which represents the maximum load car-*

*Professor of Civil Engineering, University of California

As published in the Australian *Constructional Review*, Sept. 1961.

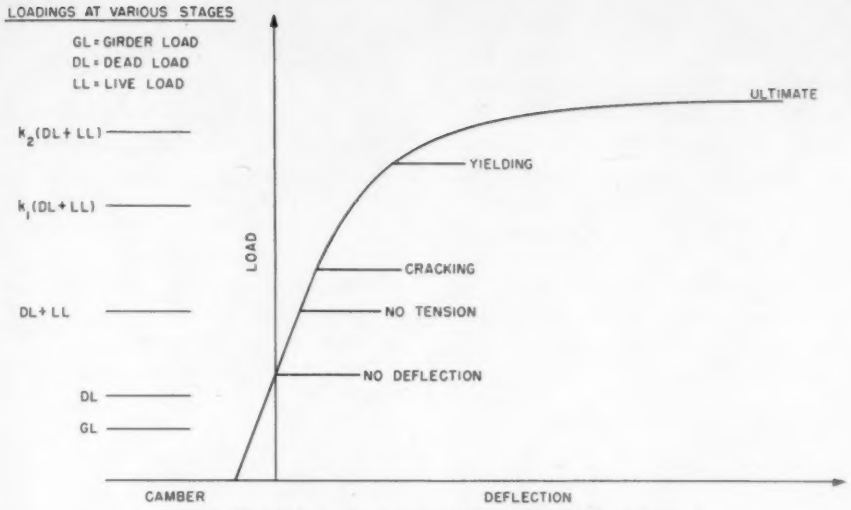


Fig. 1—Life history of a prestressed member under flexure.

ried by the member.

If the load deflection or the moment-curvature relationship is of a definite shape, it is then possible to determine all five of the above points whenever one point is known. Actually, on account of the difference in the shape of the section, the amount and location of prestressed and nonprestressed steel, as well as different stress-strain relationships of both the concrete and the steel, these load-deflection or moment-curvature relationships may possess widely divergent forms. Thus it is often necessary to determine more than one of the above five points in order to be sure that the beam will behave properly under various loading conditions.

Under the elastic stress concept, the point of no-tension or the point with a limited amount of tension is taken as the important criterion. Under this concept, Fig. 2, concrete is treated as an elastic material and a family of formulas are derived taking the shape of

$$F = \frac{Mc}{I}$$

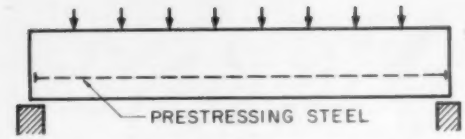
These formulas yield stresses in the beam under various loading conditions with special emphasis on the design live load.

Under the ultimate strength concept are developed a second family of formulas, Fig. 3, taking the shape of

$$M = A_s f_s jd$$

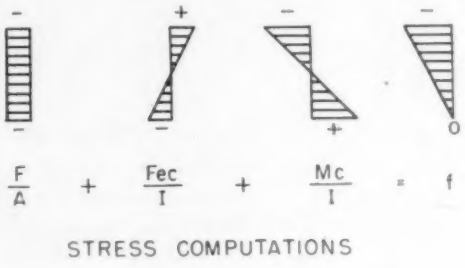
which is the familiar formula for reinforced concrete design. These formulas have also been extended to include elastic design by expressing the internal resisting moment as a T-C couple.

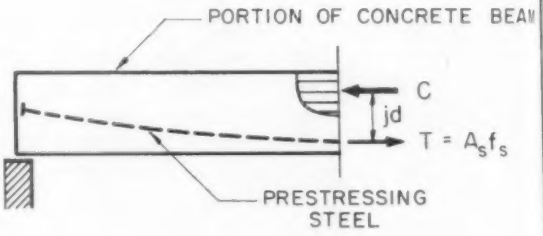
The concept of load balancing deals essentially with the first critical point, in Fig. 1, i.e. the point of no-deflection. Based on this concept the downward gravity load is balanced by the upward component of the prestressed steel. That upward component can be a concentrated force for a sharp bend, Fig. 4, or can be a distributed load for a curved cable, Fig. 5. A third family of formulas can be derived for the computation of these components. Thus for a cable with a parabolic curve, we would have a uniform upward load



Concept 1—Prestressed concrete as an elastic material.

Figure 2





Concept 2—Prestressed concrete as a combination of steel and concrete.

Figure 3

RESISTING MOMENT: $M = A_s f_s jd$

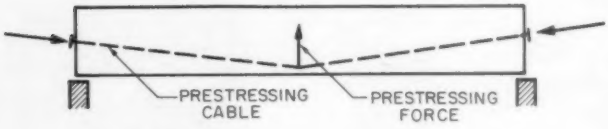


Fig. 4—Beam with bent cable.

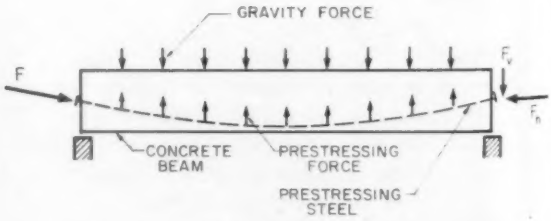


Fig. 5—Beam with parabolic cable.

FOR PARABOLIC CABLE WITH SAG h, SPAN L, LOAD w,

$$F = \frac{wL^2}{8h}$$

w, Fig. 5,

$$w = \frac{8Fh}{L^2}$$

where F = the prestressed force, h = the cable sag and L = the span length.

If the point of no-deflection, the point of non-tension, and the ultimate load are all determined for a beam, its life history is generally pretty well described. If the shape of the load-deflection relationship is known, it is necessary only to determine one of these three critical points. For convenience in design and analysis, it is possible to pick any one of the three critical points as a control. When properly designed for this controlling critical point, it may not be necessary to check for the remaining two critical points.

It should be pointed out that for different prestressed elements and structures, the best control can be obtained sometimes by using one point and sometimes by another. Since the behaviour of many elements under their sustained load is often most significant, it becomes apparent that the criterion of no-deflection could often be the best controlling point.

The Loading to be Balanced by Prestress

Using this new concept of load balancing, an important question is: what should be the loading to be balanced by the prestress? The answer to this question may not be simple. As a starting point, it is often assumed that the dead load of the structure or element be completely balanced by the effective prestress. This would mean that a slight amount of camber may exist under the initial prestress. In the course of time, when all the losses of prestress have taken place, the structure or element would come back to a level position.

Although it seems logical to balance all the dead load, such balancing may require too much prestress. Since a certain amount of deflection is always permitted for a nonprestressed structure under dead load, it is reasonable to also permit a limited amount of deflection for a prestressed one, if it would not become objectionable. However, there is a greater tendency in prestressed structures to increase their deflections as a result of creep and shrinkage. Hence the deflections should be limited to a smaller value at the beginning.

When the live load to be carried by the structure is high compared to its dead load, it may be necessary to balance some of the live load as well as the dead load. One interesting approach is to balance the dead load plus one half the live load, (DL + ½LL). If this is done, the structure will be subjected to no bending when one half of the live load is acting. Then, it is only necessary to design for one-half live load acting up when no live load exists, and for one-half live load acting down when full live load is on the structure. This idea of balancing dead load plus one-half live load, while theoretically interesting, could result in excessive camber if the live load consists essentially of transient load. If the live load represents actual sustained loading such as encountered in warehouses, excessive camber may not occur.

When attempting to evaluate the amount of live load to be balanced by prestressing, it is necessary to consider the real live load and not the specified design live load. If the specified design live load is higher than the actual live load, only a

Pereia and Luckman Architects T. Y. Lin and Associates, Consultants

Fig. 6—Union Oil bridge, Los Angeles, California.

small amount of the live load or even no live load at all should be balanced. On the other hand, if the actual live load could be much higher than the design live load, especially if the live loading would be sustained, it would be desirable to balance a greater portion of the live load. The engineer should exercise his judgment when choosing the proper amount of loading to be balanced by prestressing. This should be done while keeping in mind the satisfaction of other requirements such as elastic stress limitations, crack control, and ultimate strength.

Errors and Accuracies of the Method

A balanced load design can be achieved with considerable accuracy because both the gravity load and the prestressing force can often be predicted with precision. However, variations may be encountered so that the actual loading and the actual prestress may not be as expected. For a relatively stiff member, errors in estimating the weight and the prestress will usually be negligible. For a slender member, even slight variations may result in considerable errors in the estimation of load balancing, and either camber or deflection may result.

As is well known, the modulus of elasticity of concrete and the creep characteristics cannot be predetermined with accuracy. Fortunately, neither the modulus nor the flexural creep would enter into the picture if the sustained load is exactly balanced by the prestressing component. In other words, since there is no transverse load on the member, there will be no bending regardless of the value of the modulus or the creep coefficient.

Depending upon the accuracy desired in the control of camber and deflection, the amount of loading to be balanced must be chosen. If the limits of error can be estimated and if the significance of deflection or camber control can be assessed, it will not be difficult to design the member so as to possess the desired behaviour.

Simple Beams and Cantilevers

While the basic method for balancing a concentrated load is illustrated in Fig. 4, an actual application of this theory is shown for the Union Oil bridge, Fig. 6. This pedestrian bridge is actually made up of two bridges. (Fig. 7.) Section A-A shows columns only on the outside of each bridge while "invisible columns" are supplied at points C and D by bending the prestressing cables.

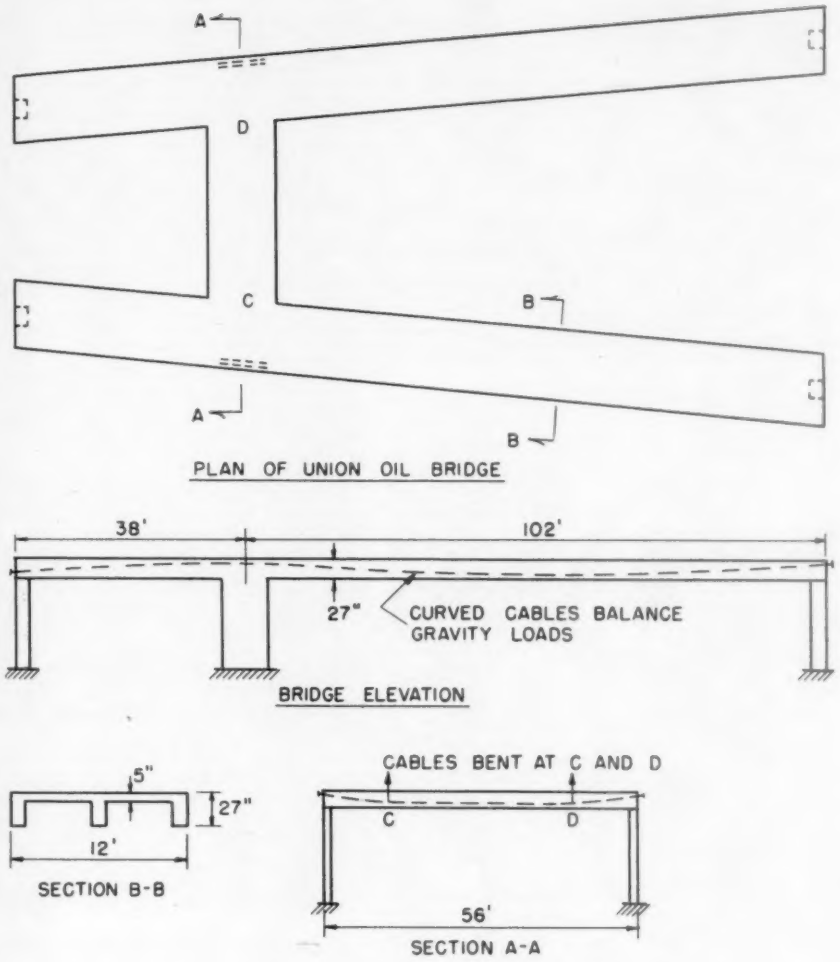


Fig. 7—Plan sections and elevations of the Union Oil bridge.

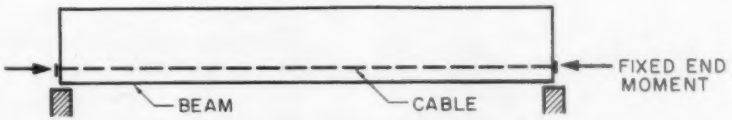


Fig. 8—Beam with a straight cable.

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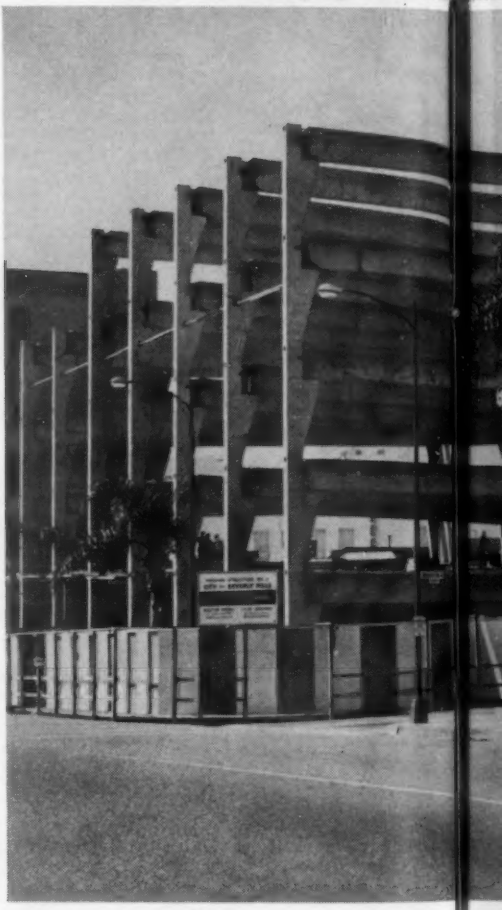
Angeles, Cal.; Consulting Engineers: T. Y. La[illegible],soci



PLASTIMENT increases workability of lightweight concrete

The five-story Beverly Hills Garage illustrated above, providing parking space for 400 cars, is a unique precast, prestressed structure. Long spans of 75 feet eliminated columns in parking areas and use of lightweight aggregate minimized horizontal and vertical loading.

Use of Plastiment Retarding Densifier increased the workability of the Ridge-

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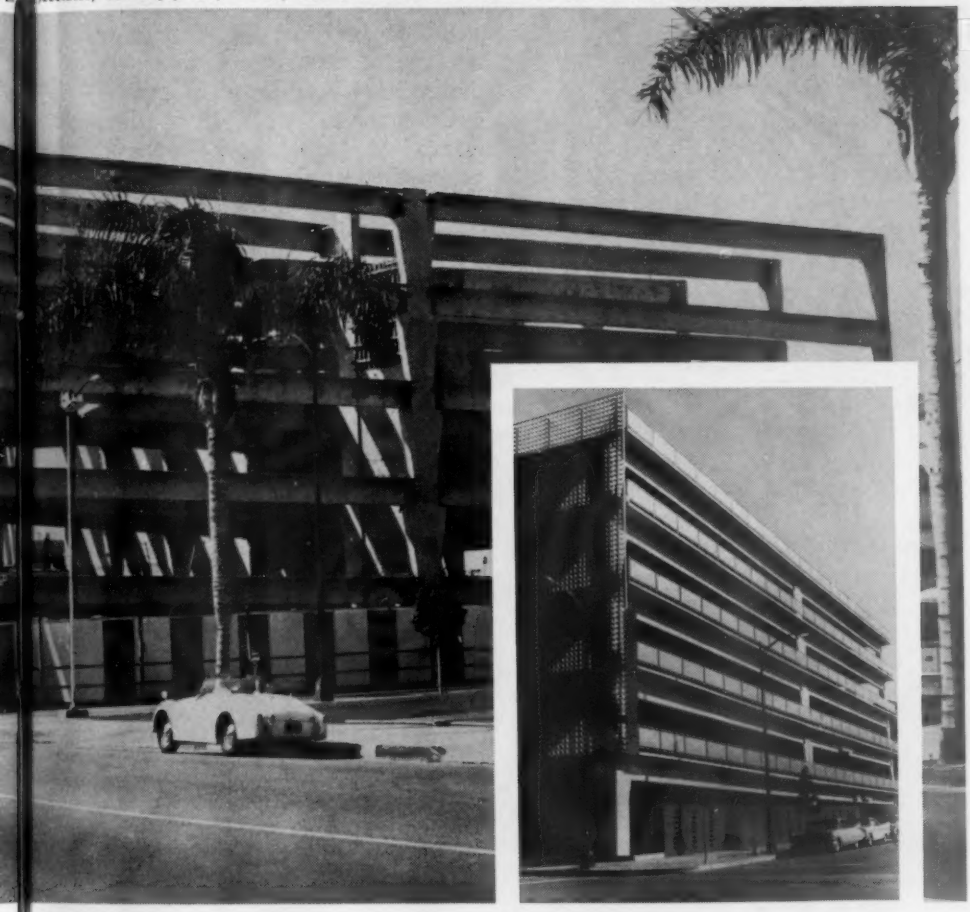

lite lightweight concrete thereby assuring a smooth, clean appearance to the exposed concrete members. Using high early cement, strengths averaged over 3,500 psi in 16 hours in the precast elements. Slab concrete placed at the site reached 3,500 psi in 2 days. 28-day strengths averaged 6,500 psi.

Mix contained 7-1/2 bags of cement per cubic yard and 2 fluid ounces of Plastiment per sack of cement.

Plastiment features are detailed in Bulletin PCD-59. Ask for your copy. District offices and dealers in principal cities: affiliate manufacturing companies around the world. In Canada, Sika Chemical of Canada, Ltd.; in Latin America, Sika Panama, S. A.

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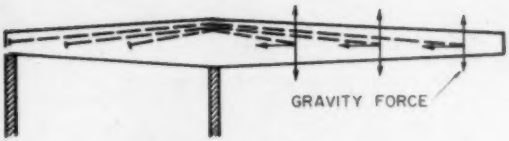


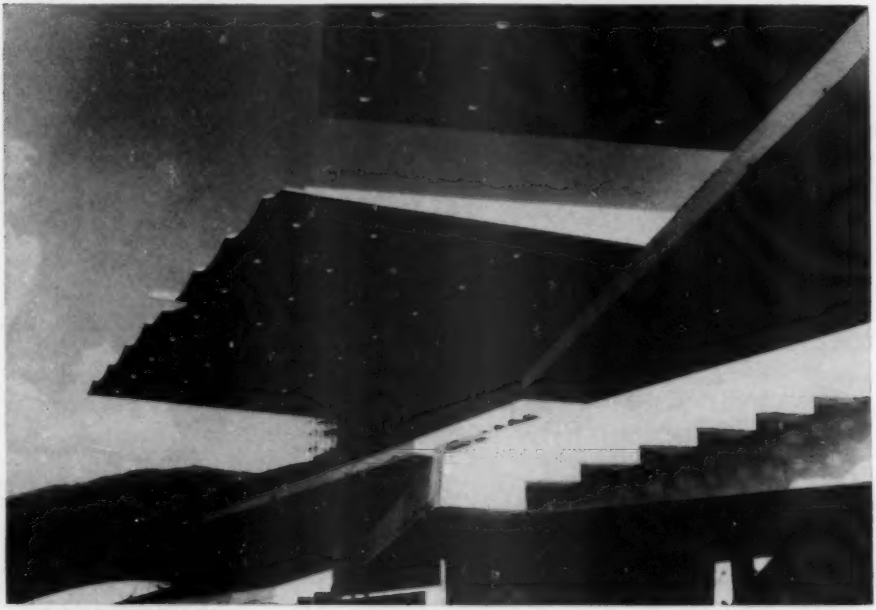
Fig. 9—Transforming a cantilever into a column.

It is more difficult to visualize the balancing of forces in a beam with straight cables, Fig. 8. It may seem that the straight cables do not tend to balance the loads. Upon careful examination, it can be noticed that a pair of fixed end moments are applied, changing the simple beam into a fixed-ended beam. While the load is not balanced along all points of the beam, it is possible to control the flexure so as to minimize the deflection.

Figure 9 shows the transformation of a cantilever into a column by prestressing. It is possible to balance the gravity force by the upward component of the prestress so that the resultant force will produce only axial stresses in the cantilever. In this manner, the cantilever is no longer subjected to bending, but is acting under direct compression similar to a column. Figure 10 shows the prestressed concrete roofs over the grandstand of Hippodromo Caracas, the national racetrack of Venezuela. These roofs cantilever 90-feet. They were made of 3-inch concrete shells, post-tensioned in two directions. The prestress in the longitudinal direction completely balances the gravity load, resulting in zero deflection at the tip of the 90-feet cantilevers. In the transverse direction, cables along the curve put the concrete into compression transforming it in to an elastic material with no cracks at all. This structure in fact represents a combination of concept 1 and concept 3 described previously.

Continuous Beams and Rigid Frames

The design and analysis of pre-



Arthur Froehlich Architect — T. Y. and Associates

Fig. 10—90 ft. Prestressed concrete cantilevers at the Hippodromo, Caracas, Venezulela.

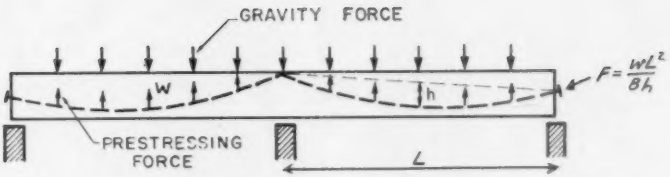


Fig. 11—Analyzing a continuous beam design.

T. Y. Lin and Associates

Fig. 12—Feather River bridge, California, Two 150-ft. spans.

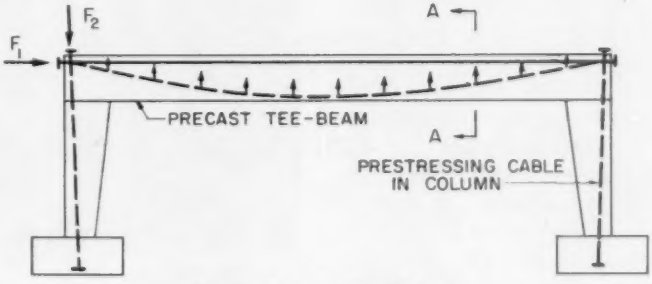


Fig. 13—Prestressed rigid frame.

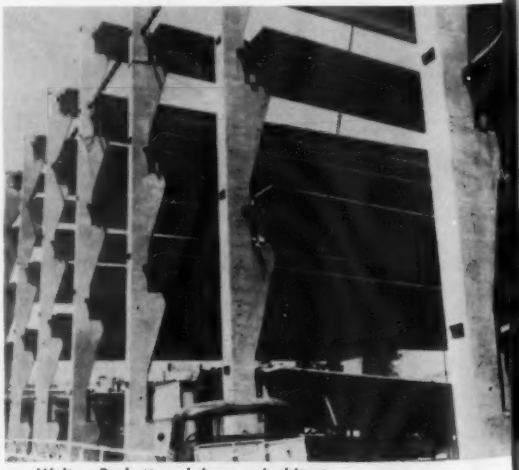
T. Y. Lin and Associates, Consultants

Fig. 14—Telecomputing facilities, Chatsworth, California.

stressed continuous beams are greatly simplified using the concept of load balancing. Figure 11 shows a two-span continuous beam. The cables in the beam exert an upward uniform force w which balances a uniform gravity load of the same magnitude. For the purpose of design, it is only necessary to determine the amount of w to be balanced. This would generally be the dead load or the sustained load. For purposes of analysis, it is only necessary to analyse this prestressed continuous beam like an ordinary elastic beam without prestress, and subject it to the unbalanced portion of the load which would be the live load or a portion of it.

Consider the two-span continuous Feather River Bridge, Figure 12, constructed in 1955. It was originally designed using the elastic concept with a criterion of no-tension. It was then checked by the ultimate strength theory and it was discovered that a factor of safety of as much as 11.2 existed. Under its dead weight, the bridge cambered up over 1-inch at midspan. If this bridge were to be designed using the balanced dead load concept, it should have no deflection nor camber under its own weight. It can then be checked for the actual elastic stresses under the design live load. If rational distribution factors were applied, no actual tension would exist. A more economical design with better behaviour would then result while the bridge would still have a factor of safety of 9 or 10.

The design and analysis of rigid frames would also be greatly simplified. In Fig. 13, the parabolic cable in the beam balances the gravity load. The eccentric end component F_1 is balanced by another eccentric force F_2 from the tendons in the columns. Thus, the rigid frame will be subjected to no bending under a



Welton Beckett and Assoc., Architects

Fig. 15—Beverly Hills garage.

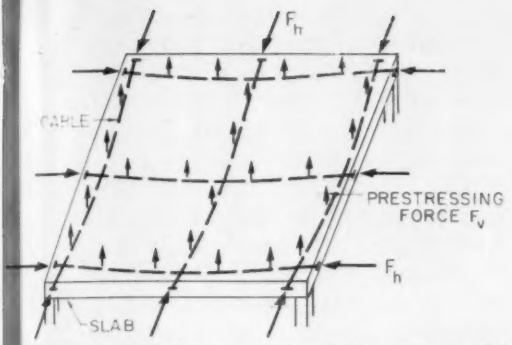
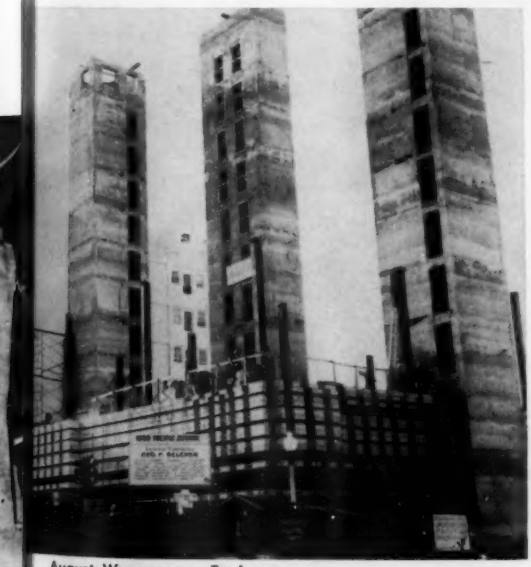


Fig. 16—Balancing gravity force transforms slab into a wall.

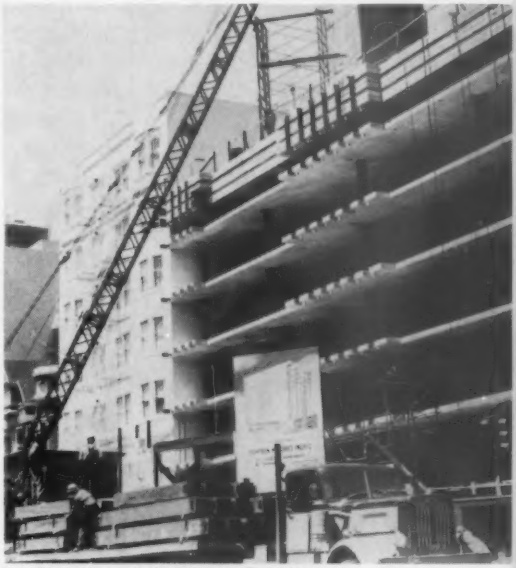
given condition. It can then be analyzed for loading variations, temperature and shrinkage stresses like a nonprestressed elastic frame subjected to relatively small loads.

Fig. 14 shows a precast prestressed concrete beam of the T-shape. This girder has a span of 120-feet. It was pretensioned with bent strands in the factory so as to almost balance the weight of the girder itself. In addition, post-tensioning strands were embedded in the concrete. After these T-girders were erected onto the columns at 20-feet centers, 5-inch concrete slabs were poured over the top and post-tensioned transversely. The post-tensioning in the slabs was designed to balance the weight of the slab and transmit load to the T's. Then the T's were post-tensioned so as to balance the additional weight of the slab, thus maintaining a practically uniform stress distribution throughout the T-section at all times.



August Waegemann, Engineers

Fig. 17—Lift slabs, Belcher Apartments, San Francisco.



T. Y. Lin and Associates, Consultants

Fig. 18—Lifting slabs into position.

There are several other interesting design features in this structure. First, the columns were also post-tensioned so as to provide rigid frame action capable of resisting earthquakes. The big holes through the webs of the T's were provided for the passage of air conditioning ducts. These holes were permissible because the girders were designed for uniform stress distribution under the dead load. The concrete slab being prestressed in two directions provides a watertight deck without roofing. This is used for parking automobiles and is believed to be a very

economical system.

The Beverly Hills Garage, (Fig. 15) with precast columns five stories high, and precast, prestressed T-beams post-tensioned together to form a multi-story rigid frame is another example of the application of this principle.

Two-dimensional Load Balancing

It is more difficult, but also more interesting to balance the loads in a two-way slab, (Fig. 16) or a two-way grid system. The analysis for such slabs and grids subjected to internal prestress could be a very difficult problem. However, if the cables balance the greater portion of the load, it is then only necessary to analyse them as an elastic slab under the action of a fraction of the load. Since we are dealing with a fraction of the load, an error in the analysis would constitute only a small variation from the actual stresses. Thus, the analyses of prestressed structures is made simpler than nonprestressed ones.

Figures 17 and 18 show a 13-story lift slab building with all the slabs post-tensioned on the ground to be lifted into position. These 8-inch slabs have a maximum span of 32-feet. They were made of lightweight concrete and post-tensioned in two directions. Here again the concept of balancing gravity loads was utilized in the design and perfectly flat slabs were obtained as a result. This type of construction enables a flat ceiling with a minimum structural depth and is very economical for apartment buildings.

For the grid system shown in Fig. 19, the cables run in two directions and exert upward and downward forces to balance the gravity load. In addition, cables can be buried in the column caps to balance the load around the periphery of the caps. Thus, a very complicated system is reduced to an ordinary elastic analysis, with the additional simplification that only a fraction of the load, rather than the total load, is being considered in the analysis.

The United Airlines Executive Office building in Chicago, Fig. 20, has a typical bay of 60-feet x 66-feet. It was post-tensioned in two directions to balance the dead load plus partitions. Again, it has practically no deflection under the sustained load and was built at a very economical price.

Three-dimensional Load Balancing

In thin shells and folded plates, the patterns of load translation cannot be easily determined. Very often it will require careful analyses—either mathematical or by use of models—to determine the magnitude and location of the prestressing force to effect a balanced design. It also happens that practical limitations may prohibit the exact balancing of the load since the cables might have to run in different directions and along different curves. However, certain simple shell structures can be post-tensioned rather readily.

Consider a cylindrical shell, Fig. 21. If the shell is relatively long, it usually behaves according to the beam theory in which case it will be possible to balance the beam action with cables having a parabolic vertical projection. If the beam action is neutralized by prestressing, the bending stresses will exist only in the transverse direction and the solution becomes greatly simplified. The California State Museum of History and Art, Fig. 22, with two spans of 75-feet each has 3-inch lightweight cylindrical shells post-tensioned with only 0.3 lb. of steel per sq. foot. The gravity load was completely balanced, the shells had no deflection

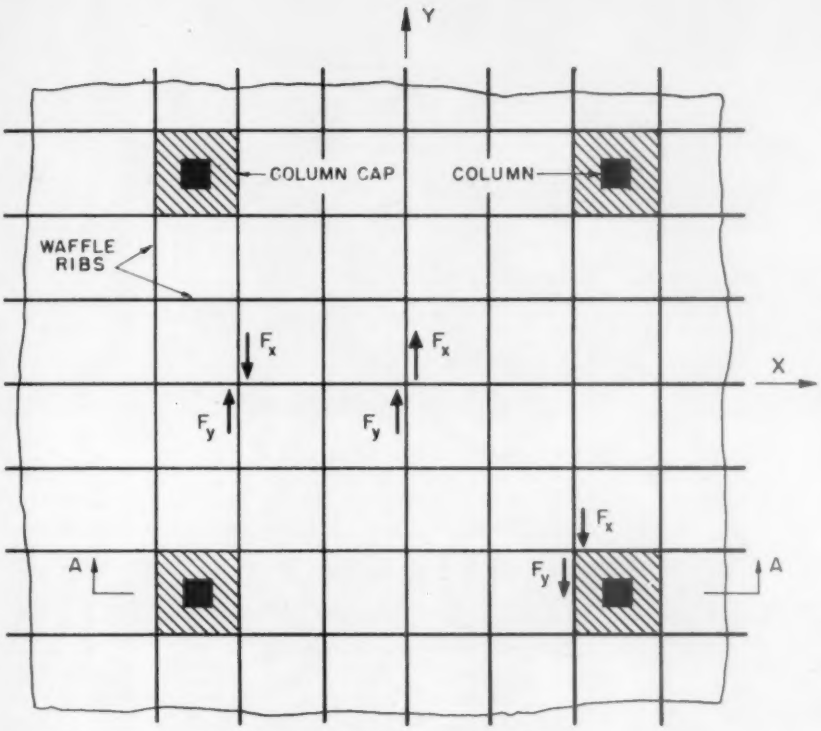


Fig. 19—Prestressed grid system.

Skidmore, Owings and Merrill, Architects and Engineers

Fig. 20—United Air Lines Building, Chicago.

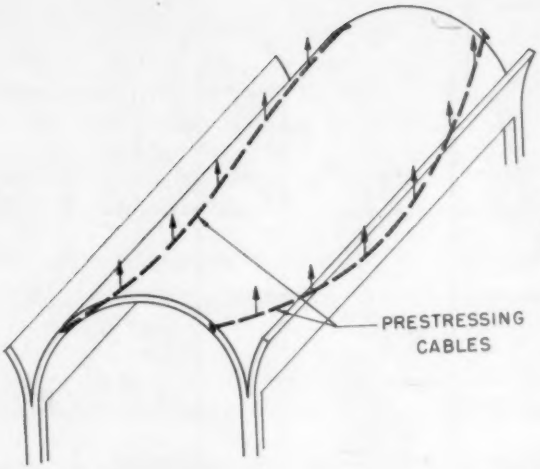


Fig. 21—Cylindrical shell prestressed for load balancing.

California State Division of Architecture

Fig. 22—California Museum of Science and Industry.

Pier Luigi Nervi

Fig. 23—Olympic Sports Palace, Rome.

Arroyo and Menendez

Fig. 24—Sports Palace, Havana.

and their secondary stresses were also minimized.

A dome shell can be balanced around the edge beams so as to control the direction of force below the edge. Fig. 23 shows a nonprestressed dome shell that carries its membrane thrust along an inclined direction into the ground. Fig. 24 shows a dome that is prestressed to divert its inclined membrane thrust into a vertical direction. Thus, the dome can be supported by vertical walls below the edge beam. Fig. 25 illustrates the application of greater prestress so as to change the membrane thrust from outward into an inward thrust, thus creating a unique design.

Hyperbolic paraboloid thin shells of the inverted umbrella type can be prestressed along their edges. Fig. 26 shows a laboratory shell of this type post-tensioned with only two wires ¼-inch in diameter along each edge. The shell was 1-inch thick and 15-feet x 15-feet square. It carried 247 pounds per square foot of sand before failure. This tremendous load capacity was possible only because of prestressing which balanced the major portion of the load.

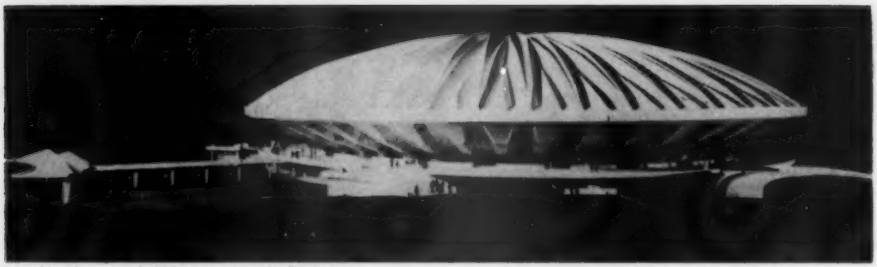
Conclusions

This concept of load balancing basically deals with a critical point in the life history of a structure or member.

Since this critical point is representative of the structural behaviour during the greater portion of the life span of the structure, it deserves more consideration than the elastic stresses for the design load or the load factors for the ultimate strength. It gives a new approach to the design of complicated prestressed structures.

Relatively simple preliminary designs can be made without many calculations. The final analysis is also simplified because a complicated problem of prestressing is reduced to an elastic analysis under the action of a portion of the live load.

When the general principles described in this paper are expanded, a new horizon will be opened for architects, engineers, and builders.



Harrison and Abbramovitz, Architects

Fig. 25—University of Illinois Stadium, Urbana.

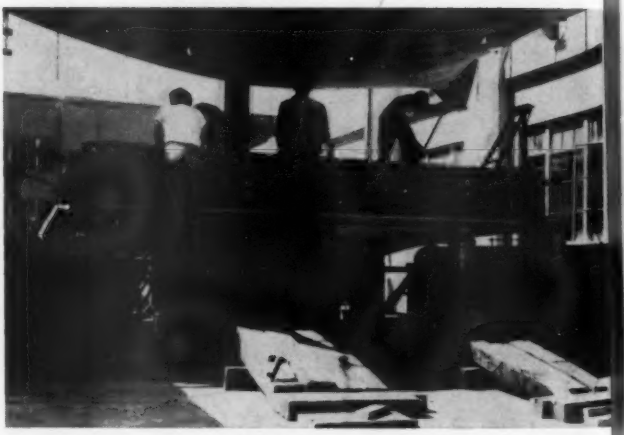


Fig. 26—Prestressed hyperbolic paraboloid.

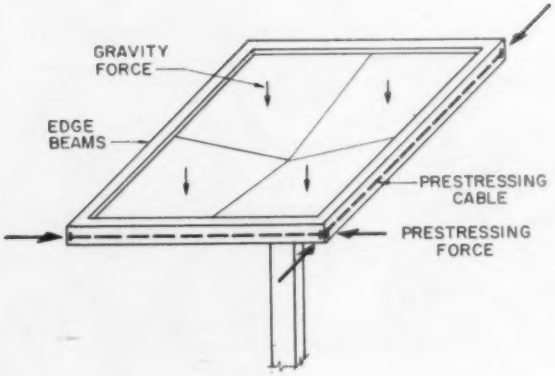


Fig. 27—Forces on a prestressed hyperbolic paraboloid.

A Monolithic Post-tensioned Parking Structure

Presented at the Prestressed Concrete Institute Convention
October, 1961

by Carroll S. Delaney, Principal*
and Neil R. Runyan, Design Engineer*, P.E.

The Denver-United States National Bank Lincoln Street Addition now under construction in downtown Denver will use more post-tensioning steel than any previous Denver building. The project will include three underground levels, three levels above grade, and eventually when completed eleven levels above ground (figs. 1 & 2). The new parking structure will be located directly across the street from the existing bank building. The basements under the parking building will extend three levels below grade. The two upper basement levels will extend under the street to connect with the basement and first floor of the existing building, thereby creating continuous banking floors approximately two blocks long. The parking floors will be served by an adjoining spiral ramp structure.

Before describing the problems connected with monolithic post-tensioning, the writers believe it worthwhile to mention briefly some interesting design features and problems created by the connection with conventional reinforced concrete portions of the building.

All the drive-up services for the Denver-United States National Bank are channelled along Lincoln Street. The bank felt that it was absolutely essential that these services not be completely interrupted. Therefore, it was necessary to build the Lincoln Street tunnel with a minimum of traffic interference during construction. Two requirements of the design were to keep the street surface construction time to a minimum and to keep one-half of the street open at all times. The solution to this was to drill and pour the caissons and the basement columns to the street level first on one-half of the street and then construct one-half of the street waffle slab. After the pouring of the first half of the street slab, the same procedure is to be used on the second half. Excavation and completion of the under-street basement will follow the surface construction without traffic interference.

The parking structure has floors 133 ft. wide by 225 ft. long. Except for some stair and elevator columns, the only columns within the interior of the building are a single line down the center of the long direction of the building, thus forming floor spans of 66 ft. 6 in. These 66 ft. 6 in. spans were framed with 24 in. deep diagrid waffle slabs supported by beams 30 in. deep. Post-tensioning was used in the parking floors and in the ex-

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Denver, Colo.

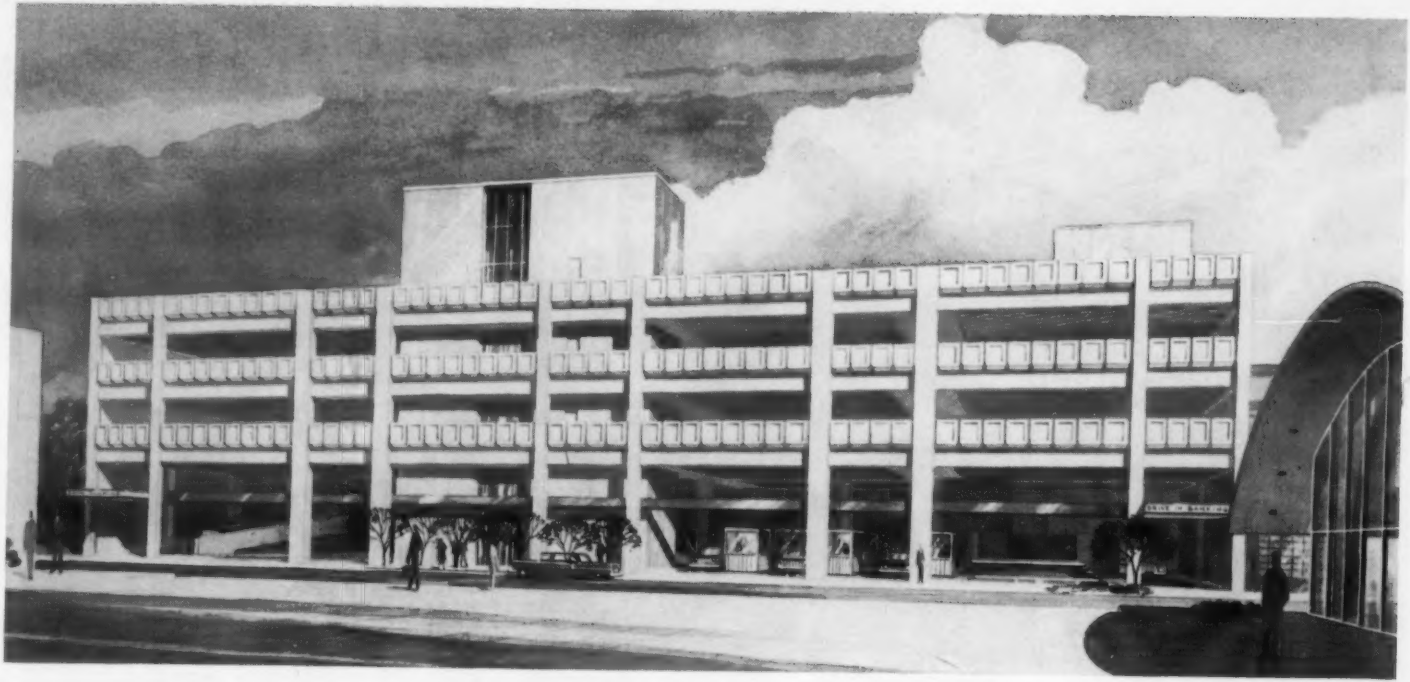

Fig. 1—Denver-United States National Bank parking structure.

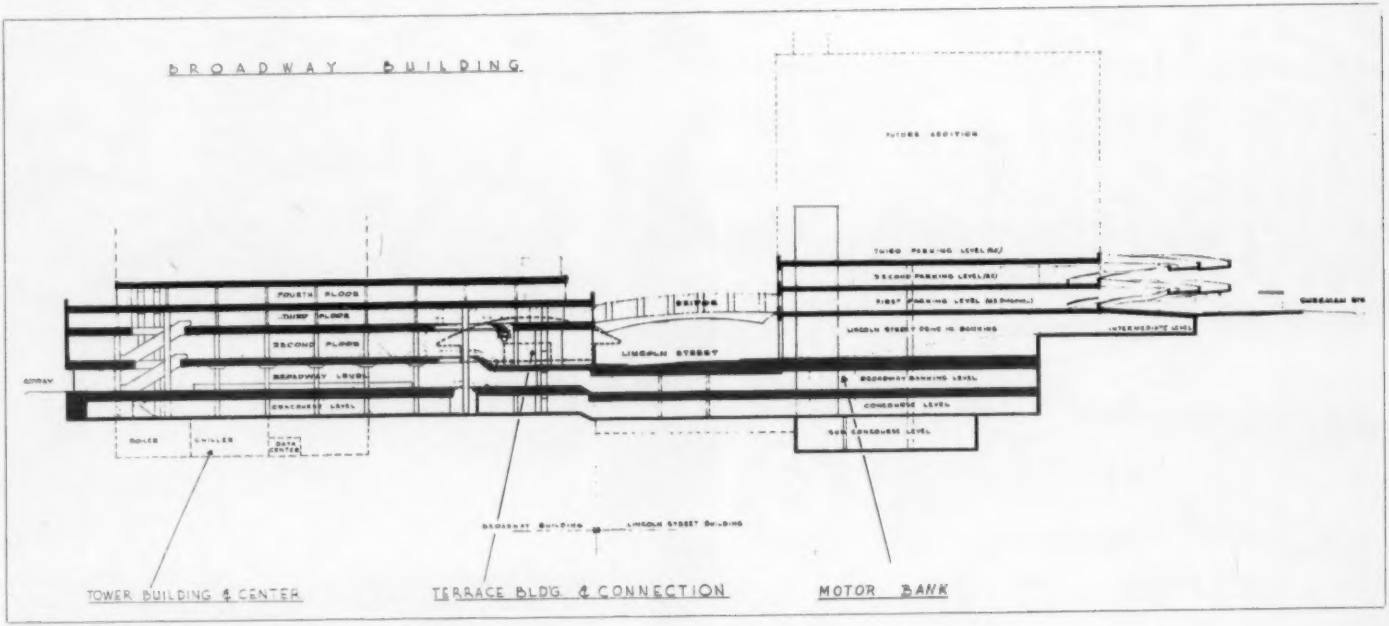

Fig. 2—Elevation of the Bank and parking structure.

terior curved ramp beam of the adjoining spiral ramp structure. Post-tensioning was required to decrease deflections in the floor slabs and beams, and to increase the torsional capacity of the curved ramp beam.

Mild steel reinforcing was desired throughout the structure to help eliminate any cracks in the concrete which might be caused by shrinkage, unusual loading, or movement caused by the stressing of other members. By using mild steel in all the beams, they became "partial prestressed" members; hence, it was not convenient to design on a stress investigation basis. The decision was made to design the flexural members by deflection and ultimate strength criteria. The deflection criterion was an undeflected floor under dead load. Live load deflection calculations were performed as a check in areas where deflection might be excessive. Almost all of the flexural members were continuous, with the number of spans varying from two to nine. To simplify the calculations involved in designing the tendons, the design procedure selected was the method of using the uplift caused by the tendon drape as an upward load on the concrete. This uplift was made equal to the dead load on the member so that no dead load deflection would occur in either a simple or continuous span. Using this approach, continuous span tendons were sized almost as easily as simple span tendons. In most members, grouted tendons were used for greater ultimate strength, although unbonded tendons were used in those cases where ungrouted tendons were adequate for ultimate strength.

The structural floor system incor-

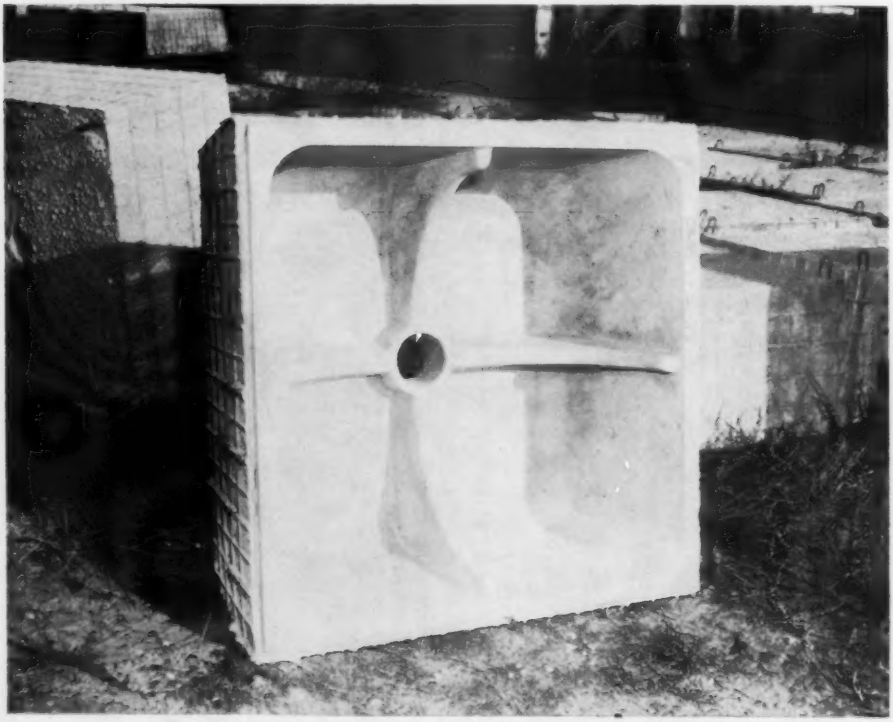

Fig. 3—Precast concrete pans for the diagrid waffle slab.

porates 10 in. wide, 24 in. deep, 45 degree diagonal waffle slab joists, and 30 in. deep beams. Pans were needed for the diagrid waffle slabs which were of greater depth than those commercially available. To provide larger pans, precast concrete pans were designed which become a part of the floor slab (fig. 3). The diagrid slab structure produces an attractive ceiling and a very stiff floor system. For efficiency it was necessary to place the beams so as to create diagrid bays of nearly equal sides. The post-tensioning of the diagrid waffle slabs caused a redistribution of the dead load to the beams different than that normally encountered in conventionally reinforced concrete. In a conventional slab the loads are distributed to the supports in approximately a parabolic manner, but in the undeflected post-tensioned diagrid slabs, the dead load is distributed approximately uniformly to the support beams. In the beam designs and in the column load calculations, it was found that excessive error was incurred when the slab live load was distributed uniformly, or trapezoidally; therefore, for the beam and column designs, the slab-to-beam loads were calculated as uniform dead loads and parabolic live loads.

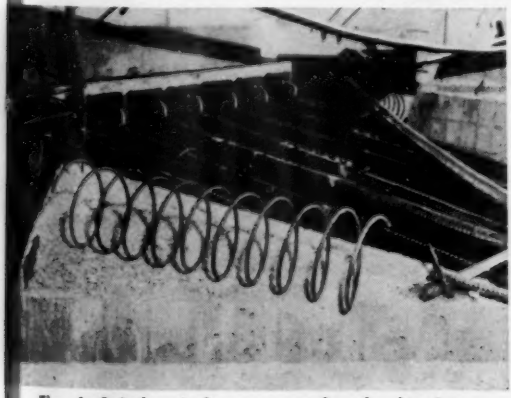

Fig. 4—Spiral reinforcement for the bearing plates.

Electronic computers were used extensively in the designs to reduce the man hours of laborious calculations. One computer program was designed to size joist tendons for the diagrid slabs. Beam shears, moments, and deflections were calculated by another computer program. A moment distribution program (available to the public from the University of Houston) helped reduce the work involved in the beam designs.

In the preliminary designs an attempt was made to design the columns for live load moments plus the moments induced in them by the shortening of the floor due to post-tensioning. This approach was not feasible for the large, perimeter columns because of the amount of moment anticipated in the floors. In fact, the bending stiffness of some of the columns would have caused them to resist almost all of the beam post-tensioning force resulting in very little beam compression.

The decision was made to pour all of the columns, except some near the center of the building, after the floor above the columns has been post-tensioned. The construction technique developed by the engineer to handle this problem entails the use of 6 in. diameter steel pipe sleeves installed in the floors at each column location to permit the pouring of the columns through the floors. In order to reduce the possibility of the column concrete settling below the bottom of the floors, the columns are to be poured in two stages. In the first stage the concrete is poured to within two feet of the beam soffit. In the second stage, which is to be poured at least three hours after the first stage, the column pour will be completed.

It was impractical to prestress some areas of the building, such as

the elevator areas. These areas of reinforced concrete were designed to seat or key into the prestressed floors after the post-tensioning had been completed. Although it would have been possible to design mild steel capable of carrying the tension caused by the post-tensioning movement that would occur, it would probably be rather difficult to avoid some unsightly cracking. The best approach to this problem seemed to be to pour these areas after the post-tensioning was complete.

In the areas where it was practical, the post-tensioning pockets were made with the top open to the top of the floor in order to reduce the construction labor involved in pouring the pockets after post-tensioning.

The reinforcement behind each tendon bearing plate consists of a 6 in. diameter spiral (fig. 4). These spirals have been used for this purpose on previous designs with success. They are easier to install and require less steel than a reinforcing bar grid.

Curved Prestressed Beam

Post-tensioning of a nearly semi-circular horizontally curved beam was a new use of prestressed concrete, as far as the writers can determine. The outside (up) ramp beam is free of columns through an arc of 147 degrees and has a diameter of 96 ft. The post-tensioning is beneficial in increasing the torsional capacity of the concrete and in decreasing the deflections. The method of using the uplift caused by the tendon drape as an upward load on the concrete does not entirely describe the effect of a tendon on a curved member. It is also necessary to consider the radial forces acting on the member. Twisting of the member will result where the tendon is above or below the shear center of the section.

Theoretically, the tendon friction loss would be extremely high in the post-tensioned curved ramp beam. A curved test beam was made in order to determine the curvature friction coefficient, μ, for the ¼ in. wire mastic coated tendons (fig. 5). This model consisted of a semi-circular curved beam containing a four-wire tendon and an eight-wire tendon. The tendons were jacked alternately from each end, with readings taken simultaneously from opposite ends, in order to determine the loss of force due to friction.

Tables I and II show the results of the friction tests.

TABLE 1
4 Wire Tendon

Jack force (kips)		% Loss
Jack No. 1	Jack No. 2	
10.	5.9	41
15.	9.1	39
20.	12.4	38

TABLE 2
8 Wire Tendon

Fig. 5—Curved test beam for determining friction loss.

Jack force (kips)		% Loss
Jack No. 1	Jack No. 2	
20.	11.2	44
30.	17.7	41
40.	27.1	32
50.	31.8	36

The curvature friction loss coefficient may be determined by assuming a "K" (wobble) coefficient and calculating the μ value from the formula:

$$\frac{F_2}{F_1} = e^{-\mu\theta - KL}, \qquad F_2 < F_1$$

The friction model tendons had the following dimensions:

$$L = 14 \text{ ft. } 10 \text{ in.}$$

$$\theta = \pi$$

Taking a friction loss value of 40% again, but assuming a "K" value of .003 gives a μ value of 0.148. Clearly the μ value is in the range of 0.15 for these mastic coated tendons. This is much lower than the recommended values generally in use today.

A 1:12 scale model of the post-tensioned curved ramp beam was constructed and tested to verify the general behavior and failure characteristics (Fig. 6). The beam was made of light weight concrete using Idealite aggregate and type III Portland cement. The 96 in. diameter model, which had a rectangular cross section of 2 in. by 5 in., carried a total uniformly distributed load of 1,914 lbs. at failure. The beam had almost reached its ultimate bending capacity when it failed suddenly in torsion at the "quarter point". The only reinforcement in the model was a single ¼ in. diameter high-strength post-tensioning wire. Briefly, the results of the model test show that the prestressing greatly increases the torsional capacity of a horizontally curved beam, and that deflections may be reduced by the use of prestressing. The analysis of the camber due to prestressisg was considerably more difficult than for straight members.

This structure emphasizes the fact that the shallower depths with longer spans are economically feasible with a monolithic post-tensioned structure. The additional stiffness and material savings usually found in continuous floor systems are possible in a post-tensioned slab and beam floor. However, many problems do occur which must be studied with care. The problem of the moments developed in the columns due to the post-tensioning deformations will occur in any monolithic concrete structure. In some structures, small concrete or steel columns can be designed to withstand these deformation moments. In tall multistory buildings it will often be necessary to relieve or eliminate these column moments by some construction procedure, such as pouring the columns after the post-tensioning of the floor above is complete, or by some type of sliding connection between the columns and beams.

A post-tensioned structure will probably always require more field supervision and inspection than many other types of construction, but certainly this is truer on today's jobs because of the workers' lack of experience with this type of construction.

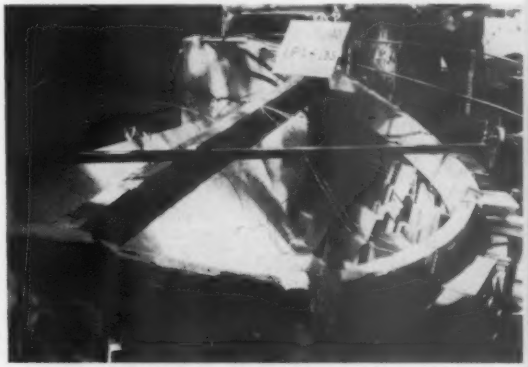

Fig. 6—Scale model of the curved ramp beam tested to failure.

Research In Prestressed Concrete At The University Of Florida And Its Practical Application In Bridge Practice

Presented at the Prestressed Concrete Institute Convention
October, 1961

By W E. Dean*

For the past 10 years construction with prestressed concrete by the Florida State Road Department, and prestressed concrete research by the Department of Civil Engineering of the University of Florida, have been continuous and complimentary activities. During this period, prestressed concrete construction has developed from a structural innovation to one with standard and well defined practices. The Florida Road Department has been the financial sponsor for several research projects, however University activity has not been limited to this sponsorship. Several research projects have been sponsored by other organizations and much research has been carried out independently by University personnel. This research has shaped and assisted developing construction practices and, in many cases, confirmed practices tentatively established.

It is not the purpose of this paper to include the entire range of prestressed research at the University of Florida. Research projects have been too numerous and their scope too broad to describe here. An appendix is included giving a complete listing of all research projects and technical papers concerning prestressed concrete conducted by University of Florida personnel to date. This paper is limited to brief descriptions of a few research projects that have been particularly significant to the practice of the Florida Road Department. Numbered references in the text refer to similarly numbered projects in the appended listing.

Of especial interest to the writer have been the numerous cases where a research project has been started with some particular object in view and, either by accident or design, has been broadened. In several instances the "fringe benefits" thus derived have been as informative as the original objective.

The normal procedure in the establishment of any structural practice is to complete many tests and much significant research before putting the new system to work. In some cases, prestressed practice has not followed this conventional procedure. The techniques of prestressing have developed so rapidly that research has often been hard-put to keep abreast of practice. In much of our work, reason and intuition have told us to go ahead and, at the same time, we set up research projects to test our theories. Thus it is, that practice has often preceded research. A major contribution of prestressed research has been in the form of review, revisions, and confirmation of rather well established practice.

In our work we have now about

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Tallahassee, Florida

reached the point that research is leading practice. With practice well established, it seems reasonable to expect that continued research will lead to broadening, refining and improving.

The following summaries are intended to show a practical and complimentary relationship between a large public works agency and a capable research organization.

Stress Losses

Our earliest sponsored research project was initiated in 1952 to evaluate actual stress losses in beams due to shrinkage and creep. These studies were continued in successive projects to about 1957. In the later studies, actual amounts of these losses were determined for eight, full sized post-tensioned bridge beams. (25)(27). Four of these beams were in the new Gandy Bridge over Tampa Bay, two were stored in the Contractor's yard and two in the University's structural laboratory at Gainesville, Florida.

Tendons in these beams were high strength bars stressed to 100,000 psi with an assumed stress loss of 15,000 psi. Recesses in the concrete were provided to accommodate gage points attached to the stressed bars. Strain measurements were commenced immediately after the bars had been stressed and periodic observations were continued over a period of more than two years.

Stress losses during the observation period are shown in Figure 1. Shrinkage differences varying with humidity are quite evident. The most humid beam location was in the bridge where the lower flanges were within 8 feet of mean low water and subject to frequent wetting from spray and waves. Stress loss in these beams averaged about 14,500 psi. The beams in the casting yard were located about 100 feet from the shore, on cribbing about 15 feet above water level and exposed to the sun. In this drier storage, stress loss was about 17,000 psi. For the driest storage in the structural laboratory at Gainesville, stress loss was about 26,000 psi.

This program assured us that our assumed stress loss was reasonable for the particular use made of these beams. For general application, the greater losses by calculation from formulae in either the BPR Criteria or Committee 323 report appear to be justified. Also the general recommendation in the Committee 323 report for an assumed loss of 25,000 psi for post-tensioned tendons appears to be justified.

7/16" Cables

Another program under study in 1955 was initiated for the principal purpose of comparing curing methods. (5)(9). At that time, the size of 7-wire strand for pretensioning was limited by Bureau of Public Roads Criteria to ⅜ in. In our own State practice we were regularly using 7/16 in. cable stressed to 18,900 lb., however this practice was questioned on Federal Aid jobs. This study was enlarged to include observations of strand slippage for concretes of various ages and strengths.

Thirty-seven beams 6 in. x 6 in. x 6 ft. were cast with one–7/16 in. strand at their centers. Eighteen of these beams were moist cured and 19 were steam cured. Stress release was applied at ages and strengths varying from a few hours and 600 psi to 7 days and 4,200 psi. Apparent end slippage of cables was measured at both ends of each beam at stress transfer. The beams

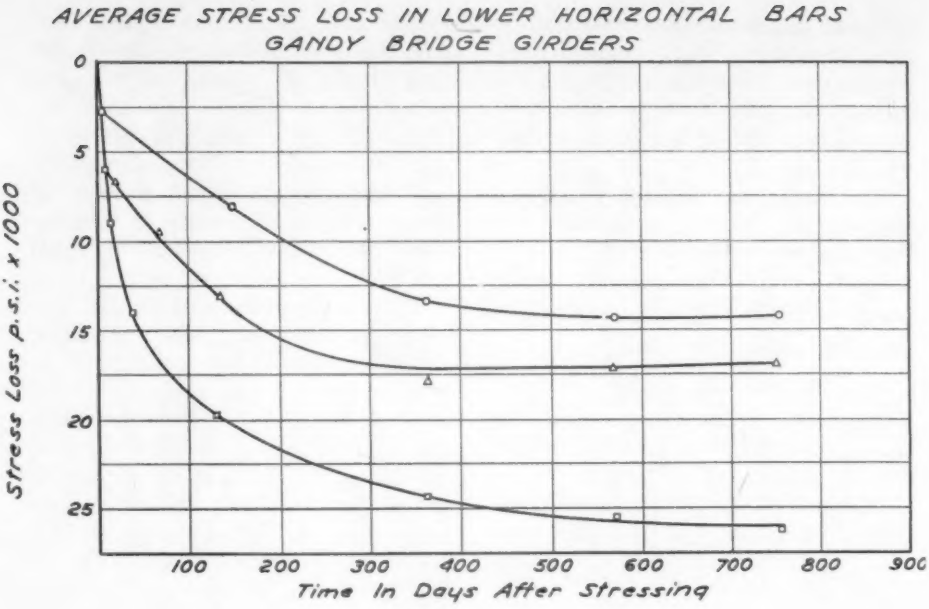
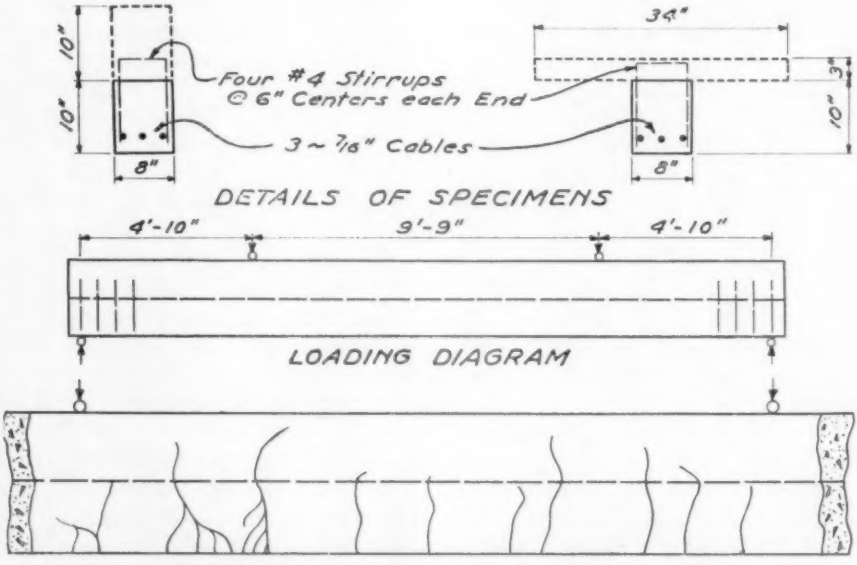


Figure 1



COMPOSITE SECTIONS IN FATIGUE

Figure 2

were then broken by center loading of a 5-foot span.

Apparent slippage of the cable at beam ends was common to all beams but for all concrete of 3,000 psi compressive strength and above, the amount of slippage was insignificant indicating full stress take-up in about 12 in. lengths at beam ends. The conclusion of principal importance to our practice for this series of tests was:

"Beams with 3,000 psi cylinder strength and 3 x 10^6 psi modulus of elasticity appeared to have reached acceptable bond, compressive and tensile strength to proceed with stress releasing."

This same research project was then expanded to include beams similar to those being used in standard bridges. Twelve "I" shaped beams, 2 ft. 3 in. deep, of a section being regularly used in Florida bridges were cast. Each beam was reinforced with 19–7/16 in. cables, each stressed to 18,900 lb. Prior to stress release, these beams were cured in three groups of four, for periods of 2, 3 and 4 days, half being moist cured and half steam cured. Cylinder strength at release varied from a low of about 3,000 psi for the 2-day moist cured to 6,000 psi for the 4-day steam cured. Upon release, cable slip measurements were made at both ends of each beam. These varied from about 0.031 in. for the 2-day moist cured to 0.007 in. for the 4-day steam cured.

Upon completion of the 2, 3 and 4-day curing periods, half of the beams were tested in bending by mid-span loading and half placed in dry storage for camber observations. A number of important conclusions were developed regarding curing methods, and resulting strengths, camber and physical properties. The fact having most bearing on Road Department practice was that "7/16 in. strands appeared to develop complete bond at a distance of 12 in. to 20 in. from the ends of beams" and in no case did slippage continue after release nor was there any evidence whatever of the ultimate capacity of the beams being limited by bond of the cables.

This series of tests convinced us of the safety of our current practice in using 7/16 in. cables with stress release at 4,000 psi. However, for further verification a concurrent project was undertaken to study the performance of pretensioned beams with 7/16 in. cables under repetitive loading. (22).

Eight beams 6 in. x 8 in. x 20 ft. were cast using 2–7/16 in. cables at 18,900 lb. This resulted in a definitely under-reinforced section. Stress release was accomplished at approximately 4,000 psi. All beams were then tested to destruction by center point loading on a 19 ft. span.

One of these beams was tested statically with an ultimate load of 3.3 times design cracking load. The other seven were tested under repetitive applications at various load levels. Results can be bracketed between the beam which did not fail at 3,000,000 applications of 1.8 times design load, to the beam that failed at 126,000 repetitions of 2.74 times design load. In no case was there any indication of end slippage of the cables or of ultimate capacity being limited by bond. A number of important conclusions resulted but the one most important to our practice was:

"The use of 7/16 in. seven-wire strand in pretensioned beams—seems feasible."

Following these tests our use of 7/16 in. cables was no longer ques-

tioned by any approving agency.

Composite Construction and Shear Keys:

There is extensive literature on composite construction consisting of steel beams with a poured-in-place concrete deck, and many tests have been performed on various means of bonding the two elements. Little has been published on composite construction using concrete for both elements. In our earlier designs of precast beams and situ-cast deck, mechanical bonding devices consisted of stirrups extending out of the beam into the slab and raised or depressed keys on top of the beam. At best these keys were construction nuisances. Raised keys are apt to interfere with placing of slab steel and depressed keys are perfect trash catchers.

In initiating its prestressed practice, the Florida Road Department tested to destruction, statically, three full sized composite bridge sections consisting of a 48-ft. prestressed concrete beam with a 6-ft. wide slab cast on top. The elements were bonded by shear keys and stirrups. In each case failure occurred in the center third of the beam from a combination of flexural and diagonal stress. In no case did failure of bond between the two elements contribute to failure of the specimen; in fact there was no evidence of the slightest distress along the plane of joining.

A fourth such test was made without shear keys but with the top of beam moderately rough. Results were the same.

In the fifth test the top of the beam was given a smooth trowelled finish. The results remained as before. This test was reported in the technical press (Engineering News-Record June 7, 1956) and we concluded that if sufficient steel was provided to develop the vertical component of joint stress, the horizontal component could be taken by bond of the concrete, provided the bond stresses were within the limits ordinarily used for bond of reinforcing bars. This reasoning would eliminate shear keys in most instances but was questioned by our reviewing agency because all of our tests had been static and results under repetitive loading were desired.

In 1957 a project was initiated at the University of Florida to study several aspects of composite construction consisting of precast-prestressed members with a cast in place top (16) under repetitive loading. A principal objective was observation of the joint between the two elements for which no shear keys were to be provided.

Four beams 20 ft. long were used for these tests. Each beam consisted of a precast-prestressed rectangular section measuring 8 in. x 10 in. reinforced with three—7/16 in. cable tendons with a cast-in-place top. For two of the beams the cast-in-place portion was 8 in. x 10 in. to make a rectangular member 8 in. x 20 in. with the horizontal joint at mid-height. The cast-in-place concrete section for the other two members was a T slab 34 in. wide and 3 in. deep. All of the precast sections had their upper surfaces trowelled smooth. The only mechanical ties between elements were 4—½ in. stirrups at 6 in. centers at each end. No grout or bonding agent was used between elements. The contact area was merely dampened before casting the upper portion.

After curing, these beams were subjected to repetitive quarter point loading. All beams were tested to failure by various combinations of cycles and overloading ranging from

1.7 to 2.6 times design live load. The most severe application of horizontal shear between the precast and cast-in-place elements was the beam in which this shear varied from 180 to 320 psi for 866,000 applications. In no case did horizontal shear contribute to failure of the specimen, in fact there was no case in which the slightest sign of distress appeared along the junction of the two elements.

Figure 2 shows details of the beams used in these tests with typical load application and crack pattern. In cases where cracks reached the joint between elements, they extended straight through with no tendency to follow the joint.

Results of this series of tests consisted of several important conclusions regarding differential shrinkage, relative moduli of elasticity, and bond between elements. The most helpful conclusions affecting our practice were:

"None of the composite beams showed any failure along the plane of contact.

"The natural bond between the two concretes and the presence of four shear ties at each end of the beam provided a composite action strong enough to withstand repetitive overloads."

Following these tests our practice in composite beams has been to provide extended stirrups to develop the vertical component of shear between elements. Tops of beams are left moderately roughened and shear keys have been generally eliminated.

½ Inch Depressed Strand — End Zones — Neoprene Pads:

Following full acceptance of 7/16 in. strand, similar tests were instigated in 1957 to justify the use of ½ in. strand. (18). The original project in this series was quite similar to the last described study on 7/16 in. strand and results were fully as conclusive. In 1959 and 1960 another series of tests were conducted on beams with ½ in. strand with the scope of study considerably broadened. (10).

In 1957 and 1958 the use of depressed or harped strands in heavy bridge beams became quite general among Florida producers. This practice had ample precedent in other areas but some concern was expressed at the possibility that the sharp angles at points of cable harping might cause stress concentrations which would limit fatigue capacity. At the same time, the AASHO specifications for prestressed bridge beams required the use of end blocks, although there was much doubt as to their necessity in pretensioned members and much precedent for their elimination. This research project was accordingly designed to study fatigue loading in beams with ½ in. harped strands and without end blocks.

Six I shaped beams, 26 in. deep and 41 ft. long, as shown in Figure 3, were tested. Prestressing was accomplished by 8–½ in. strands at 25,200 lb. of which two were straight and six were harped. Points of cable harping were 4 feet each side of mid-span and points of loading were 5 feet each side of mid-span. Web reinforcing consisted of single #4 bars at 6 in. centers thruout the beam lengths.

All beams were subjected to repetitive loading to failure varying from 210,000 applications of 1.86 times design load to 2,500,000 applications of 1.58 times design load. In all cases failure was taken as a 35% loss in "Spring Constant" or the ratio of design load to center deflection. In the case of these

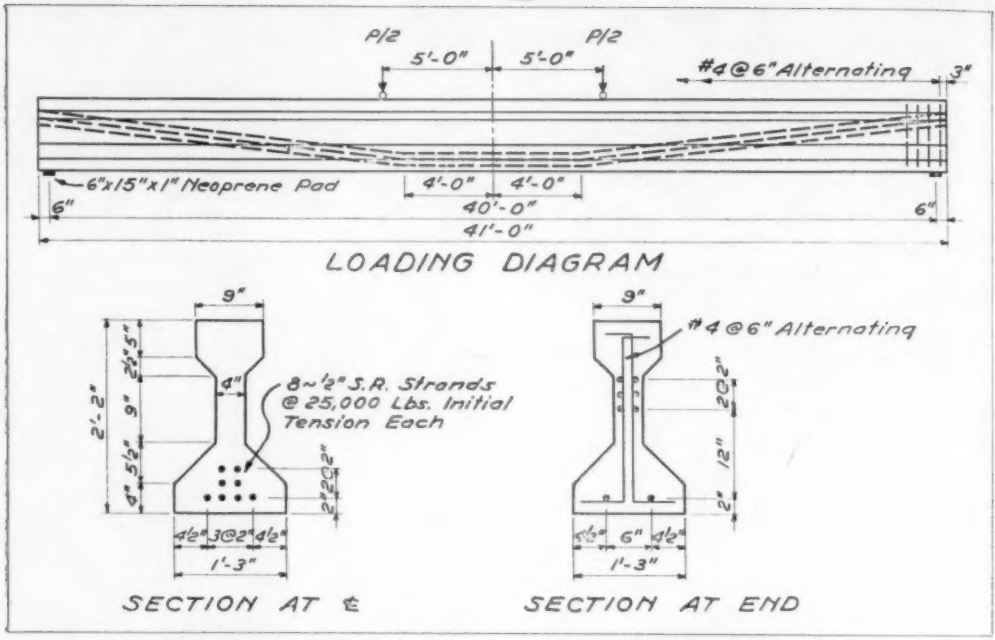


Figure 3—General details of specimen beams with depressed strands for fatigue testing.

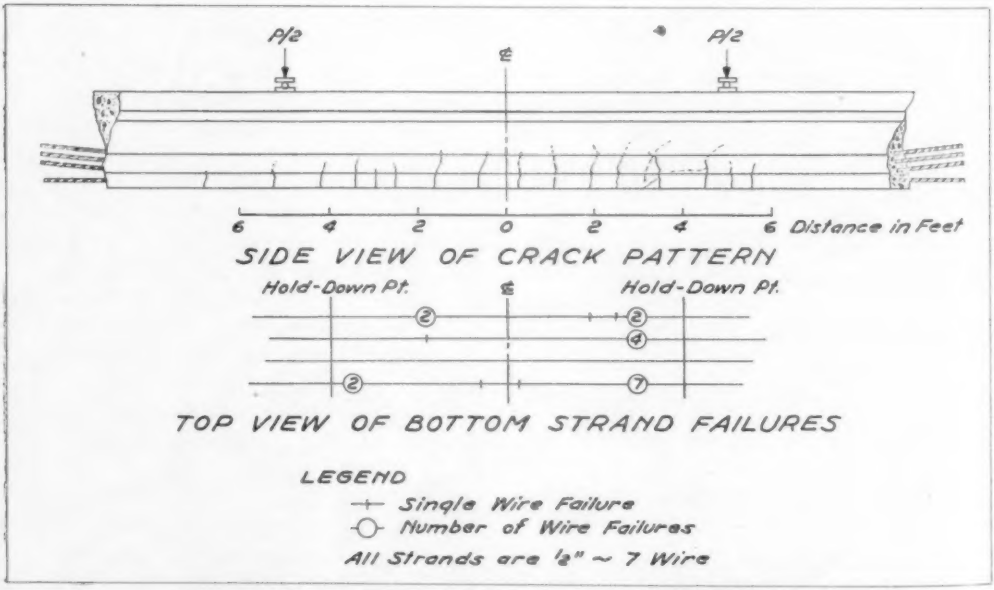


Figure 4—Crack pattern and strand failures of beams with depressed strands in fatigue tests.

beams this corresponded to a permanent set of about 0.25 inches.

A typical pattern of cracking and strand failures in the loaded zone is shown in Figure 4. In the six beams there were no failures of a strand or even of a single wire at hold down points.

In nearly all of the beams, some hairline cracks were observed in the web at span ends. A typical pattern of end cracks is shown in Figure 5. None of these end zone cracks for any of these beams extended in magnitude during the testing which signified that lack of end blocks did not reduce flexural strength.

Stated conclusions of this series of tests included:

"All the wire or strand failures occurred at locations of cracks in the concrete and no strand failure was observed at or near the hold down points."

"The omission of end blocks did not have any apparent detrimental effect on the general behavior or fatigue life of the girders."

During this series of tests all beam ends were supported on 70 Durometer neoprene pads 6 in x 15 in. x 1 in. thick. Incidental information was developed on the performance of these pads under repetitive loads up to 250 psi. Early in these tests one beam was furnished with a raised form mark across the bottom flange in the bearing area. This form mark almost centered the bearing pad in the long dimension. With about 1,000,000 load applications this pad split directly under this form mark and required replacement.

The pad at the other support was used thruout the entire series of tests withstanding more than 15,000,000 load applications. At the conclusion of the series this pad showed no damage whatever, the only physical change being a slight increase in hardness.

From this we conclude that a specification requiring accurately finished bearing areas to eliminate local stress concentrations is essential for use with these pads.

Current Research:

Current practice of the Florida Road Department in prestressed concrete is limited to precast members for which weight is an important consideration in span limitation and transporting. Some Florida producers have used light weight concrete for building members but so far our lack of knowledge of shrinkage,

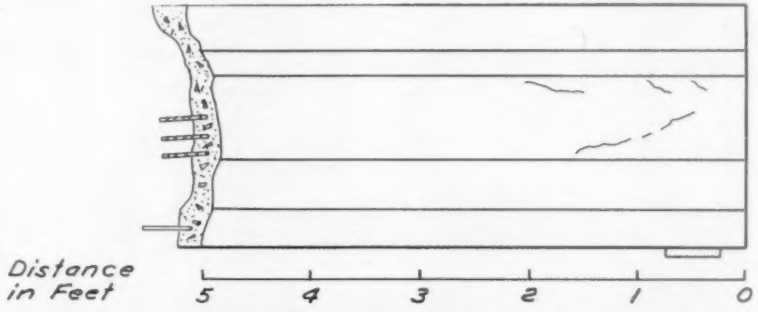


Figure 5—Typical crack pattern at ends of beams with depressed strands in fatigue tests.

creep, and modulus of elasticity of light weight mixes has restricted their use in bridge members.

A program of study sponsored by the Florida Road Department is now in progress at the University of Florida to evaluate these physical characteristics for light weight concretes using aggregates which are readily available. From this research we hope to develop data that will enable us to predict stress losses and camber and deflection values which must be known before we can safely use light weight concretes.

Equipment and Acknowledgements:

It will be noted that a major portion of the research reported herein has involved repetitive loadings. The structural laboratory at the University of Florida has two fatigue machines with capacities of 10,000 lb. and 50,000 lb. General details of the machines are shown in accompanying photographs (Figures 6 & 7). Both were designed and built by University personnel. Each machine is of a constant-deflection type consisting of a rocker arm connected to an adjustable eccentric, operated by an electric motor through a suitable reduction to give load applications of about 140 per minute. The load-deflection relationship is established by a dynamometer in the rocker arm on the smaller machine and a proving ring in the larger. Frequent load checks and adjustments of the eccentric were done in all fatigue tests to keep the load within one or two percent of desired values.

As indicated in the Appendix, prestressed research at the University of Florida has involved the joint the Civil Engineering Faculty, graduate and undergraduate students. The Florida Road Department and producers and users of prestressed products have been assisted and benefited by the dissemination of knowledge resulting from these studies. Limitations of this paper preclude acknowledgement of the contribution of each individual involved, however particular mention and thanks of the Florida Road Department and the Prestressed Concrete Industry in general are due Prof. Ralph W. Kluge, Head of the Civil Engineering Department, and Dr. Alan M. Ozell, Research Professor in Civil Engineering, who have directed and actively participated in most of this research.

APPENDIX

Research Reports and Technical Papers on Prestressed Concrete by University of Florida Personnel

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3. Ozell, A. M., "Choice of Sections—Camber," Jan. 1958.*
4. Zia, Paul, "Ultimate Strength Design," Jan. 1958.*
5. Ozell, A. M. and Givens, W. D., "Curing Methods and Duration Studies of Pretensioned Units," Part 1, TPR-2, Eng. & Ind. Exper. Sta., Univ. of Fla., Mar. 1956.
6. Zia, Paul, "Ultimate Strength of Slender Prestressed Concrete Columns," TP No. 131, Eng. & Ind. Exper. Sta., Univ. of Fla., July 1957.
7. Ozell, A. M. and Jernigan, A. M., "Some Studies on the Behavior of Prestressed Concrete Columns," TPR-3, Engr. & Ind. Exper. Sta., Univ. of Fla., July 1956.
8. Ozell, A. M., "Prestressed Concrete Design," Bul. No. 74, Engr. & Ind. Exper. Sta., Univ. of Fla., Aug. 1955.
9. Ozell, A. M., and Givens, W. D., "Curing Methods and Duration Studies of Pretensioned Units," Bul. No. 82, Engr. & Ind. Exper. Sta., Univ. of Fla., Aug. 1956.

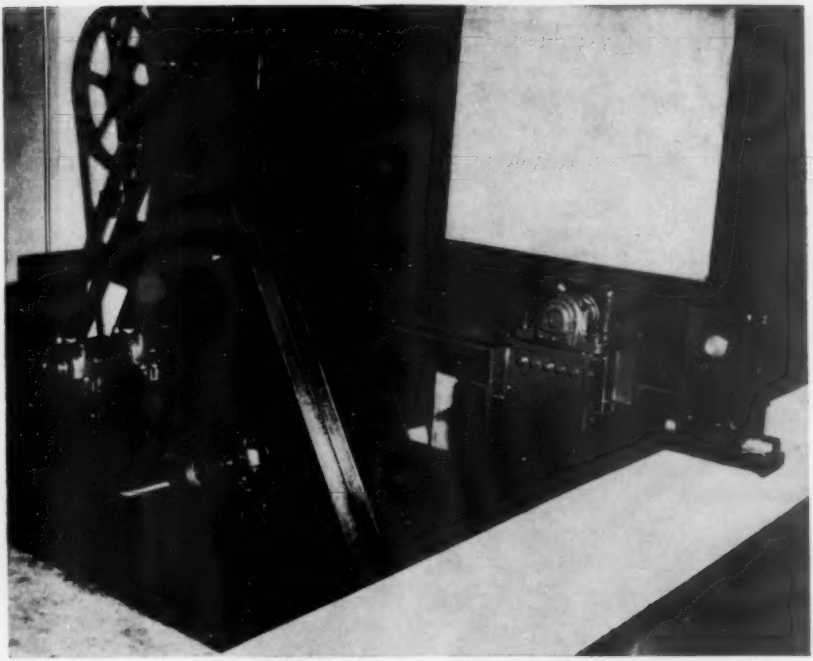


Figure 6—10,000 lb. capacity Fatigue Machine.

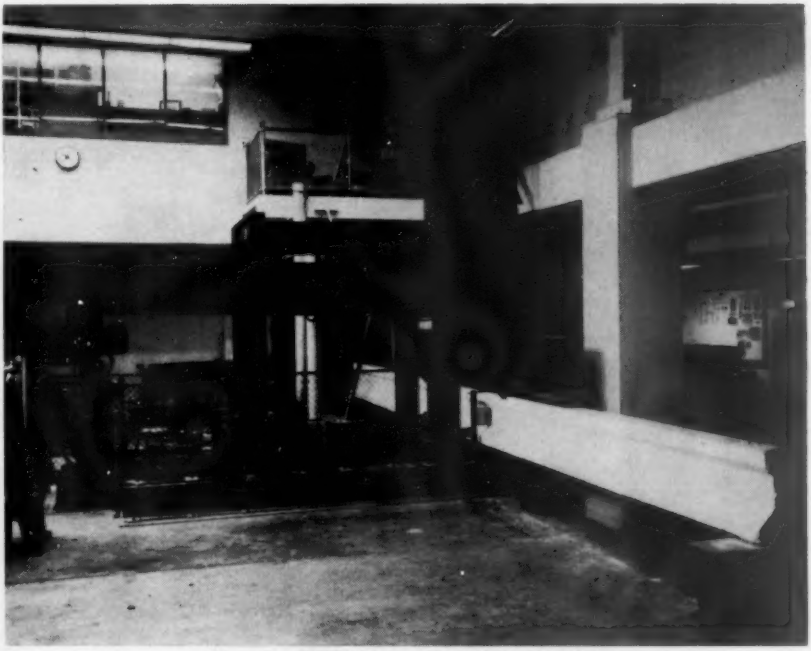


Figure 7—50,000 lb. capacity Fatigue Machine.

10. Ozell, A. M., and Conyers, A. L., "Fatigue Tests of Prestressed Concrete I-Beams with Depressed Strands," Jan. 1961 (Unpublished. Submitted to Fla. SRD.)
11. Carter, R. H., "Stress Distribution in the Ends of Prestressed Concrete Beams," June 1952. (Unpublished. Submitted to Fla. SRD.)
12. Branson, D. W. and Ozell, A. M., "Camber in Prestressed Concrete Beams," 1960. (Unpublished. Submitted to Fla. SRD.)
13. Dean, W. E. and Ozell, A. M., "Static Testing of a Composite Prestressed Concrete Bridge Section Constructed Without Shear Keys." Results published in Engineering News-Record June 7, 1956 under title "No Shear Keys are Needed Here."
14. Branson, D. E. and Ozell, A. M., "A Report of Differential Shrinkage in Composite Beams," P.C.I., Dec. 1959.
15. Lofroos, W. N. and Ozell, A. M., "The Apparent Modulus of Elasticity of Prestressed Beams Under Different Stress Levels," P.C.I., Sept. 1959.
16. Ozell, A. M. and Diniz, D. F., "Composite Prestressed Concrete Beams Under Repetitive Loading." P.C.I., Mar. 1958.
17. Ozell, A. M., "Discussion on: Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete, by Nordby and Venuti," A. C. I., Mar. 1958.
18. Ozell, A. M. and Diniz, J. F., "Fatigue Tests of Prestressed Concrete Beams Pretensioned with ½" Strands," P.C.I., June 1958.
19. Ozell, A. M. and Heasley, W. D., "Design of Prestressed Concrete Harbor Structures," P.C.I., June 1958.
20. Zia, Paul, "Ultimate Strength Design; Advanced Prestressed Concrete Design," Eng. Prog. at Univ. of Fla., Engr. & Ind. Exper. Sta., Univ. of Fla., Apr. 1958.
21. Ozell, A. M., "Behavior of Simple-span and Continuous Composite Prestressed Concrete Beams," P.C.I., (2 parts), June 1957.
22. Ozell, A. M. and Ardaman, E., "Fatigue Tests of Pretensioned Prestressed Beams," A.C.I., Oct. 1956.
23. Ozell, A. M. and Diniz, J. F., "Comparison of Prestressed Concrete Codes and Specifications of Various European Countries," P.C.I., June 1957.
24. Straubel, C. A. and Ozell, A. M., "Some Studies on the Deflection of Prestressed Concrete Beams," P.C.I., Mar. 1957.
25. Kluge, R. W., "Field Studies of Prestress Loss Due to Shrinkage and Creep," P.C.I., June 1958.
26. Ozell, A. M., "Discussion on: 'Behavior of One-Story Reinforced Concrete Shear Walls Containing Openings', by J. R. Benjamin and H. A. Williams," A.C.I., June 1959.
27. Kluge, R. W., "Prestressed Concrete Research at the University of Florida," P.C.I., Sept. 1956.
28. Zia, Paul, "Preliminary Design of Prestressed Concrete Beams," P.C.I., May 1956.
29. Marshall, P. W., "Behavior of Prestressed Concrete Under Dynamic Loading," P.C.I., Dec. 1960.
30. Zia, Paul, "Research on Torsion of Prestressed Members," P.C.I., Sept. 1960.

*These appear in Bulletin Series No. 96, Engr. & Ind. Exper. Sta., Univ. of Fla.

Red Deer River Bridge

Presented at the Prestressed Concrete Institute Convention
October, 1961

By: George Adam[1] and D. de Wolff[2]

Introduction:

In planning extensions to north-south highway #36 located approximately 100 miles east of highway #2 joining Calgary and Edmonton, the Bridge Branch of the Alberta Provincial Department of Highways found it necessary to locate a bridge across the Red Deer River at a point 8 miles north of Duchess. At this bridge site the river runs at right angles to the highway. The requirements dictated a structure 750 feet long with a 30 ft. deck and a 2 ft.–6 in. wide curb on each side.

A complete study of the most economical structure for this site indicated that the bridge should be designed with five equal 148 ft. spans to minimize the number of interior piers. As these spans were considerably longer than any previous spans using precast prestressed girders, the Bridge Branch proceeded with the design of a continuous built-up steel girder section. These girders were approximately 7 ft. 6 in. deep with 4 girders per span spaced at 9 ft.–0 in. centers. The tenders called for the supply of all structural steel, bearings, diaphragms, f.o.b. jobsite. Con-Force Products Ltd. commissioned Structural Engineering Service Ltd., Consulting Engineers, to prepare a simple span alternate design using 4 lines of prestressed concrete girders set at 9 ft.–0 in. centers. (Fig. 1) When tenders were opened, the prestressed alternate was considerably lower than the lowest steel bid; and after revised quotations were obtained to include for supply and erection of all materials, the prestressed alternate still provided the most economical structure. Based on lowest initial cost and minimum future maintenance costs, the decision was made in favor of the prestressed girder alternate on June 1, 1959.

Tender Data:

INITIAL TENDER–Supply only of girders, bearings and diaphragms, and horizontal bottom chord bracing, f.o.b. site.

Low steel fabricators .....$206,476.
Prestress alternate .....$178,000.

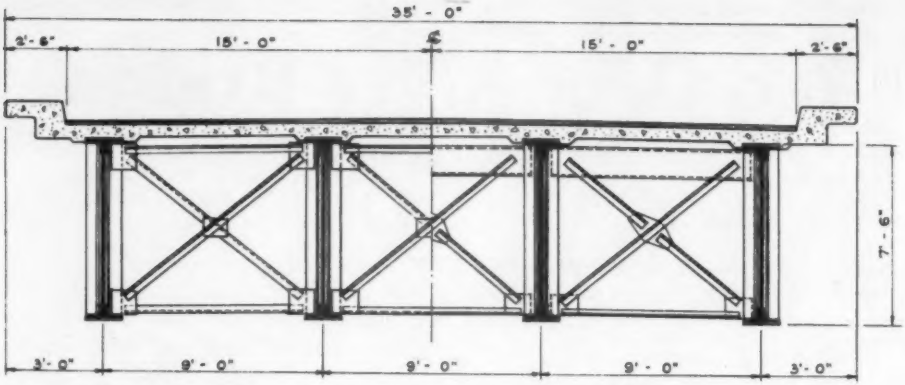
FINAL TENDER–Supply and erection of girders, bearings, diaphragm, and horizontal bottom chord bracing.

Low steel bid ..$229,139. (17.6¢/#)
Prestress alternate $223,000. ($198./yd.[3])

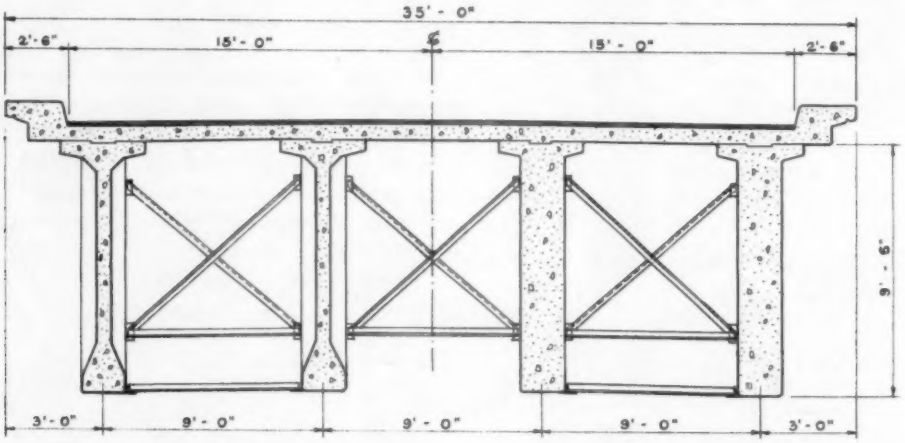
Design:

With span and spacing determined, and the design live load of AASHO H20–S16 specified, the de-

[1]Con-force Product, Ltd.
[2]Structural Engineering Services, Ltd., Calgary, Alberta, Canada



• CROSS SECTION • STEEL GIRDER DESIGN •



• CROSS SECTION • PRESTRESSED CONCRETE ALTERNATIVE •

Fig. 1—Bridge alternates—cross sections.

sign of the girders was routine.

For the trial section, the depth was selected on the basis of a depth-span ratio of 1:15, this ratio having been determined by previous experience with similar requirements. From the trial depth and the required ultimate resisting moment, based on 2 (MDL + MLL), it was determined that 14, 12/0.276 in. diameter wire Freyssinet cables would be required. A web thickness of 7 in. was selected as being the practical minimum.

The bottom flange was proportioned to suit the desired arrangement of cables keeping in mind the practical requirements for concreting a slim section of this depth.

The width of the top flange was selected from a width-span ratio of 1:42 in keeping with proportions of previously built long members.

Minor adjustments were made in the proportions of the trial section as the analysis proceeded with the final cross-section being as shown. (Fig. 2)

Principal tensile stresses at working loads were very low, but under the ultimate loading, shear was a consideration. The shear reinforcing used was ⅝ in. diameter two leg stirrups at a maximum spacing of of 18 inches.

Due to the considerable height of the girder, deck, and curb, and to conform with clause 1.6.67 of the current AASHO specifications, it was necessary to consider the effects of transverse wind loading. A system of lateral bracing in the plane of the bottom chord of the girders was designed to resist one half the total wind force. Both this lateral bracing and the transverse diaphragms, which were located at 24 foot centers consisted of light rolled steel shapes.

In keeping with the original steel design, typical rocker bearings were designed for a reaction of 259 kips.

Construction Schedule:

Tenders for the general contract on this project were then called based on the supply and erection of the girders; bearings and diaphragms were to be by others. On July 2, 1959, a contract was awarded based on the following construction schedule:

(a) July to October, 1959—Girders cast and stressed at jobsite.

(b) July 1959 to January 1960—Approach fill, piers and abutments completed.

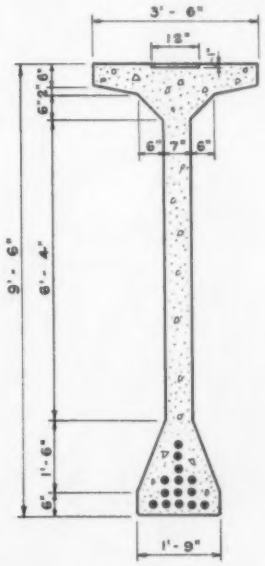
(c) January to March 1960—Girders, bearings and diaphragms erected in place.

(d) April to August 1960—Deck and superstructure completed.

This schedule enabled the general contractor to drive piles and construct piers and abutments under minimum river flow conditions. It also made it possible for erection crews to handle the girders during the winter months when ice and frozen ground permitted transportation of almost unlimited loads with a minimum of problems.

Manufacturing Girders:

Production crews were faced with the problem of casting 20 girders 148 ft.–2¼ in. long, 9 ft.–6 in. high and weighing approximately 112 tons each. Also included in the girder were the mild steel reinforcing cage, cast-in shoe plates, inserts, weld plates, and 15 Freyssinet 12 wire 0.276 cables. Two 12 ft. sample sections of a typical girder were cast in new metal forms at our Calgary plant using aggregates hauled in from the jobsite to provide information on the most satisfactory method of handling the forms, reinforcing cages, and concrete. These trial sections confirmed that cross-sectional dimensions could be kept within ± ⅛ in. of detailed dimen-



• GIRDER CROSS SECTION •

Fig. 2—Typical prestressed beam cross-section.

sions and that concrete poured with a 3 in. slump could be vibrated to provide a dense concrete around all of the post-tensioned cables in this girder. As these 112 ton girders were to be erected with two sixty ton cranes, accurate control of the overall weight of each girder was extremely important. In addition, to confirm that normal grouting procedures were satisfactory, a standard 150 ft. long cable complete with wires and metal tubing was assembled and pressure grouted at our Calgary plant.

At the site, construction crews cleared several acres of land situated above summer flood levels 2,000 feet south of the bridge site. A concrete slab 160 ft. x 120 ft. with heavy edge beams on the 120 ft. side and with sleepers placed in the long direction was prepared to serve as a casting bed for the 20 large girders required in this project. Short lengths of 2 x 4's were then nailed to adjacent sleepers to support a 2 in. thick timber base, complete with chamfer strip, running in the 160 ft. direction. The first half of the metal form was set up tight against this wood base, wedged in place, and securely braced to the front edge of the slab using steel bars with adjustable turnbuckles. Shoe plates at ends of girders and connection plates were set in place and the reinforcing steel cage, previously fabricated in sections, was assembled. The cage was then tack welded together, and supported in the correct location on this base. A one yard dragline on tracks was used to handle metal forms and sections of reinforcing steel cage. Lengths of flexible tubing complete with 12–0.276 in. diameter wires were then inserted at the end of the cage and guided through accurately located control stirrup loops attached to reinforcing steel cage. End bulkhead units containing anchorage cones were then set in position, and hose connections to cones were completed with standard Freyssinet sleeves. The high tensile wires extended through the anchorage cones at each end of the girder. The outside form was then set up tight to the base. Steel clamps were used at the top and bottom of the girder form to insure that girder cross section dimensions remained constant during the entire casting cycle.

The 56 yards of 5,000 psi concrete used in each girder was mixed at the jobsite using 2–16S drum type mixers fed by standard bin and batch field type equipment. A front end loader delivered aggregates to the storage bins, and a tractor shovel with extended arms was used to deliver concrete from the mixer to the forms. Temporary working platforms were set up, cantilevered from the forms at the most suitable height for production crews. The concrete was poured commencing at one end and kept on about a 20° slope as the form was filled. When pouring was complete and the top surface finished the entire section was covered with tarps for at least one night. Production crews poured two girders the first week and three girders each week thereafter until all twenty girders were cast (see Fig. 3). Curing was obtained by keeping the girders covered and water soaked for 5 to 10 days after pouring. Test cylinders were taken to determine standard 7 day and 28 day strengths on all girders.

The two top cables in each girder were stressed with hand equipment as soon as possible after casting to prevent shrinkage cracks from forming. Later a special crew moved in

to the jobsite from the manufacturer's Calgary plant to complete the stressing and grouting of the 14–12 wire 0.276 Freyssinet cables in each girder. The use of a portable generator and power stressing equipment enabled this special crew to complete stressing, grouting and pouring of protective concrete end cap around the exposed ends of the anchorage cones in 25 working days. The grout injected around the stressed wires in each cable was composed of high early cement and water with no other additives. The girders initially cambered about 1 in. rising clear of the wooden base at the centerline and compressing the base at each end. After stressing, the entire dead weight of the girder was carried by the heavy edge beam cast with the poured-in-place casting bed.

Erection:

While the methods and procedures used in casting these girders in the field were comparable with normal plant practice, the problems associated with lifting, transporting and erecting these girders in the most economical manner required careful planning and the use of special equipment.

During the casting of girders in the field, rectangular holes were formed in the top flange two feet in from each end on both sides of the end block. Directly under these holes a 4 in. diameter pipe section was cast horizontally in the end block. It was a simple matter to fabricate a short lifting bar with long channel straps. This was dropped through the rectangular deck holes and accepted a horizontal lifting pin which went through the reinforced bottom ends of the straps and through the cast-in 4 in. pipe section. An initial lift on each girder was made with a mobile 60 ton crane to remove the girder from the form base and line it up in the general direction of the bridge.

Specially fabricated frames were then used to lift the girder and transport it over the frozen ground and ice to a point where the girder could be erected in place on its bearings. These frames straddling each end of the girder rolled on two 18 x 24 scraper tires on each side of the girder on a pair of

Fig. 3—Completed girders awaiting prestressing.

Fig. 4—Specially fabricated transporting frames straddling end of a girder.

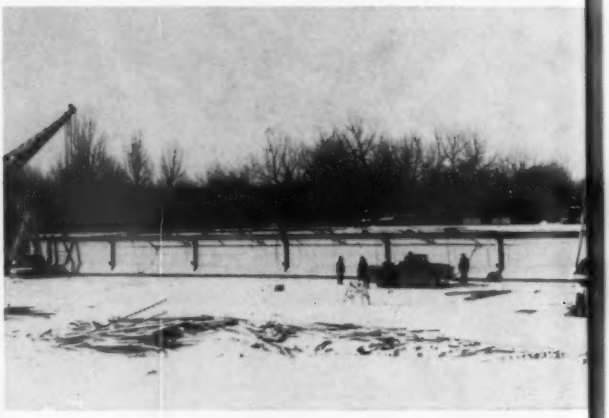


Fig. 5—Preparing a girder for movement with transporting frames and dozers at each end.

Fig. 6—Moving a girder into position for erection.

axles. (see fig. 4) To achieve stability in this lifting and travelling frame, horizontal pipe braces from the axles on each side of the girder and a heavy rigid connection from the top of the frame were all attached to the front of a TD-24 dozer. A multi-pulley fixed block was attached to the top of each frame and a movable block complete with lift hook was suspended over the girder from a 14 part line. This line led from the top pulley to the power winch on the dozer enabling each dozer to raise one end of the girder approximately 16 in. The dozers and lifting frame units were used to move the girders from the casting slab to locations beside the piers. (see fig. 5) To provide the girder with additional lateral stability, a 10 ft. wide pipe truss was fastened to the top flange while it was moved over 2,000 feet of rough terrain to the bridge site. (fig. 6)

Girders were cast in the required order and orientation to permit each girder to be moved to the site and erected in place. This was done working from east to west, with the furthest span of the bridge first, and progressively completing the structure one span at a time. After the girders had been moved to the base of the piers they were lowered onto temporary supports while the dozer units and travelling frames returned to the casting yard. The girders were then placed on metal fixed and expansion bearings using 2 cranes each with 60 ft. booms and a rated capacity of 60 tons (fig. 7) As the cranes were operating very close to their maximum capacity, some girders had to be moved again and placed on temporary blocking to enable the equipment to get close enough to the piers to complete erection with a single lift from the last storage point.

Diaphragms were made up of two vertical 3½ in. x 3½ in. x 5/16 in. angles securely fastened to the girder. Two diagonal angle cross braces and a horizontal bottom angle were bolted together during erection to stabilize girders and were finally welded after all the girders were erected in place. In addition, lateral bracing was provided to tie bottom chords of girders 1 to 2 and 3 to 4 (equivalent to the bracing shown for steel girders in the original tender drawings. (fig. 9) These 4 in. structural tees were also initially bolted and finally welded in place after girders were all erected. All bearings, girders, diaphragms and bottom chord bracing members were erected in place by March 10, 1960.

Observations:

As transportation of prestressed concrete girders exceeding 120 ft. by road or rail is either expensive or impossible, many design engineers fail to consider the possible advantages to be gained in considering girders cast in the field. Excellent members can result by using competent plant personnel with years of experience in the efficient production of post-tensioned concrete girders. When local terrain will permit the movement of heavy loads and high capacity mobile cranes are available, and bridge foundation conditions are reasonably good, prestressed girders can, for many bridge projects, provide all of the advantages normally associated with short span girders used in North America.

Fig. 7—Erecting the 112 ton girder with two 60 ton cranes.

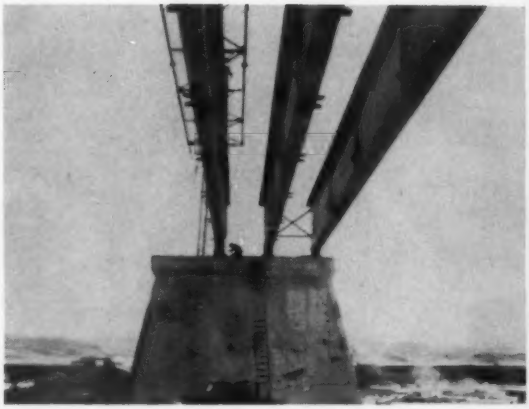


Fig. 8—Special lateral stabilizing frame used during erection is seen in this view.

Fig. 9—Lateral bracing and diaphragms are shown here.

Fig. 10—Graceful lines of completed structure are emphasized in this photo.

The Martin P. Korn Award

Past President Peter J. Verna presented a bronze plaque to Col. Martin P. Korn at the Fifth Annual PCI Convention. This plaque symbolized the establishment of the annual Martin P. Korn Award.

The Martin P. Korn Award of the Prestressed Concrete Institute was established at its Fifth Annual Convention at Miami Beach in honor of Martin P. Korn, first Executive Secretary of the Institute. This Award will be presented annually to the author of the best paper published in the PCI JOURNAL.

COMPETITION RULES

I The Award consists of $100.00 in cash, with an appropriate engraved certificate. In case of more than one author, the cash will be divided, but each will receive a certificate.

II All original papers presented to the Institute and published in the JOURNAL during the year for which the Award is given, are open to the Award provided that such papers have not been previously contributed in whole or in part to any other associations, nor have appeared in print prior to their publication by the Institute. Papers to be considered for this year's Award are those that appear in the December 1961 JOURNAL, and all 1962 issues prior to the Annual Convention.

III The Award, with certificate, is awarded to the author, or authors of a paper which shall be judged worthy of special commendation for its merit as a contribution to the advancement of prestressed concrete. The ceremony conferring the Award will be held at the 1962 PCI Convention in New Orleans, La., Sept. 23-28.

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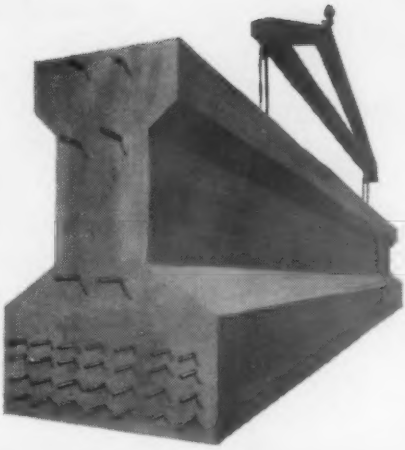
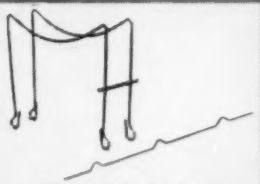
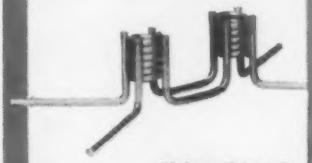


Richmond Strand Deflection Inserts for prestressed concrete beams and girders. In addition to the standard line of inserts, special units can be manufactured to suit individual needs.



Richmond Chairs and Spacers for void supports and spirals in prestressed piles and beams.



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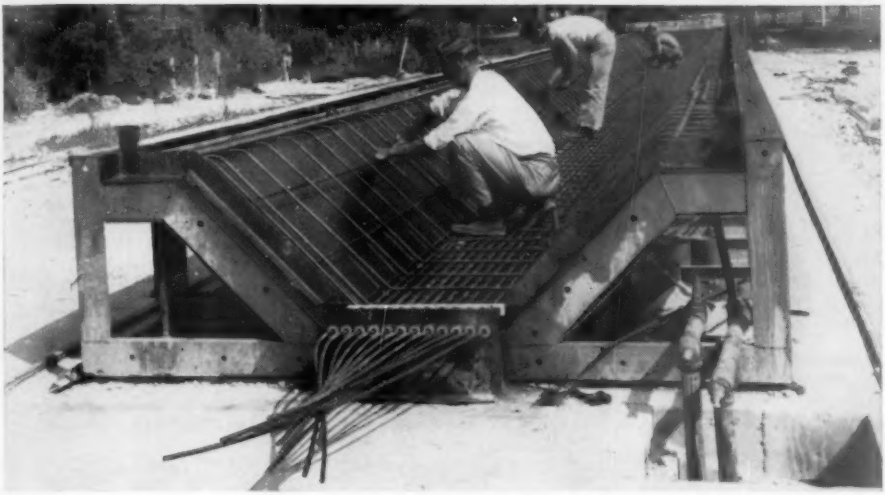
New folded-plate joining technique eliminates built-up roofing



⅜-in. Bethlehem stress-relieved strand is placed on 2-in. centers at bottom of form which was specially designed by Perma-Stress, Inc. Finished member is 8-ft wide, 28-in. high, and 3½-in. thick. It has been tested for 60 psi roof loading, three times the design load.

Bethlehem Stress-Relieved Strand used in Florida prestressed concrete school-building program

This handsome new school is one of many being erected by Volusia County, Florida. Although the dimensions vary for each school, prestressed concrete folded-plate roof elements are common to most. The use of similar units saved thousands of dollars for the county.

Another important saving resulted from a new joining technique developed by the architect, W.R. Goman & Associates. Perma-Stress, Inc., who fabricated the folded plates for seven schools in their nearby Holly Hill plant, built-in a raised lip along the upper longitudinal edge of the member. Each unit is installed tight against the adjacent plate, and a flash cap is placed over the raised portion. Since the entire roof is waterproof, there is no need for built-up roofing. The method is also less expensive than using cast-in-place concrete between plates to make a continuous rigid roof.

These attractive economical structures are fire-resistant. The schools go up in a hurry, too. So far, all have been completed well within the time limit of 185 calendar days (no provision for bad weather).

Perma-Stress, Inc. uses Bethlehem induction-heated stress-relieved strand. Like other fabricators, they find it flexible and easy to handle. And they get the same dependable mechanical properties in every reel. Call your nearest Bethlehem sales office for full details.

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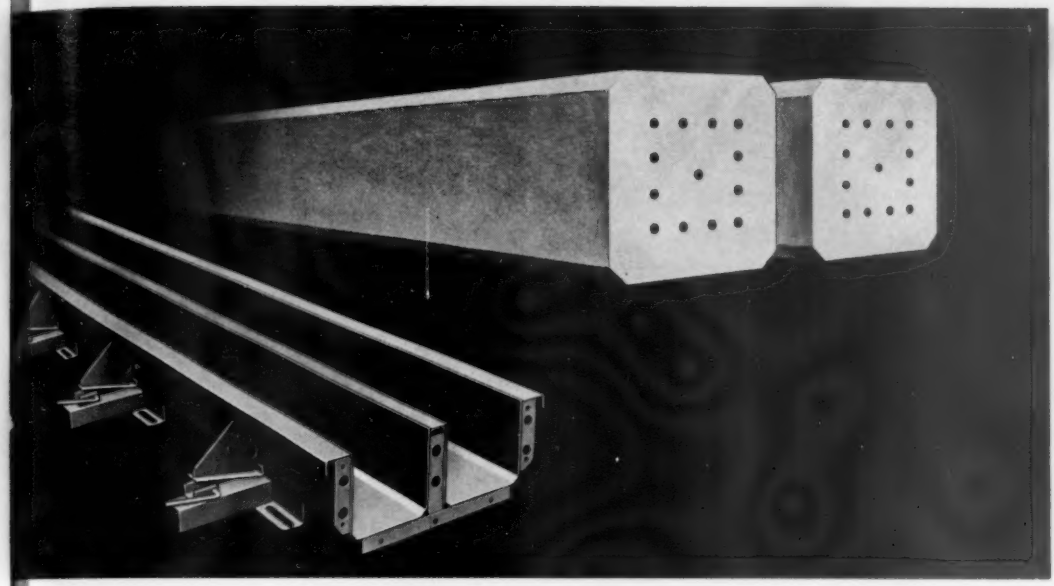
CORRECTION

The name of Mr. E. H. Ratz, Candidate of Technical Sciences, Moscow Institute of Reinforced Concrete Products, should be listed as a co-author with Mr. B. Skramtaev of the proceedings paper "Electrothermic Method of Pretensioning Bar Reinforcement of Precast Reinforced Concrete". The article was published in the PCI Journal Vol. 6, No. 3.

This issue of the PCI JOURNAL contains the first set of Proceedings Papers from the recent 7th Annual PCI Convention. Succeeding issues will contain all the papers presented at the General Sessions.



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